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PERTINENT ELEMENTS OF EXPRESS HIGHWAYS

by

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Presented before the Students

of the

Bureau of Street Traffic Research
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At the moment there appears to be a possibility that the highway industry of the United States is on the threshold of an era of express highway development. Before this development gets under way it is imperative that the fundamentals entering into the design of such highways be authoritatively established on a firm and satisfactory basis. The greatest weakness of past highway design has been the inability of design to conform, even reasonably, to the operating potentialities of the motor car; and to provide for the weaknesses and shortcomings of the driver. This has been the cause of an enormous monetary loss in physical investment due to obsolescence, untold suffering and sorrow and a staggering economic waste due to accidents.

Today, many engineers freely admit the failings of the past, but at the same time there is a reluctance to "face the facts" and to actually approach design so that the highway will be suitable for the forces which are being applied today and to care for loads which will surely be applied in the near future. To illustrate: A moving automobile weighing 3,000 pounds is a projectile with a destructive force of 160,000 foot pounds at 40 MPH, 492,000 foot pounds at 70 MPH and 1,000,000 foot pounds at 100 MPH. These figures bring to mind the age old poser of the irresistible force meeting the immovable object. Although such problems are being given lip service, engineers are shrinking from meeting the situation squarely and courageously, while, as a matter of fact, there are a number of basic features which have not been satisfactorily solved and about which an almost complete ignorance must be confessed.

I propose to discuss these problems, to set forth the factors of which we are ignorant, to indicate some of the fallacies not commonly recognized, to demonstrate that design thinking must be enlarged and further, to arrange the various component elements which comprise a completed highway project in their proper light based upon experience gained on the Pennsylvania Turnpike.

Engineering discussion over the past five years has rather definitely indicated the broad features which should be incorporated in a modern express highway. These are:

(1) The separation of the streams of traffic flowing in opposite directions to reduce the possibility of head-on collisions and side-swipe accidents.

(2) The elimination of all highway and railway grade crossings, so that there will be no cross traffic.

(3) The provision of wide smooth stable shoulders level with the pavement in order that vehicles may park completely off the paved area and to provide "elbow room" in cases of traffic emergency or mechanical failure of the vehicle.

(4) The provision of two lanes of pavement in each direction in order to reduce the menace due to slow moving vehicles, to eliminate the necessity of encroaching on the lanes of opposite direction traffic when passing and so that there may be no delay in passing.

(5) The provision of traffic lanes of sufficient width to adequately care for the safety, comfort and convenience of all classes of traffic.

(6) The elimination of all frontage, "wildcat" commercial development and local farm entry along the right-of-way and the complete control of that right-of-way.

(7) The exclusion of pedestrians.

(8) The selection of an adequate design speed and the arrangement of the design elements to conform to that speed.

(9) The provision of moderate rates of grades in order to increase the speed of large heavily loaded trucking units so that rear-end collisions between fast and slow vehicles are minimized; and to assure safety in descent during icy weather.

(10) The provision of uniform and consistent operating conditions.

(11) The provision of specially designed access facilities at reasonably long imtervals with acceleration and deceleration lanes in order that vehicles may enter and leave the main highway with reasonable safety and a minimum of traffic disturbance.

These features have been accepted by hard headed engineers because accident statistics clearly indicate that the mere provision of them will automatically eliminate the causes of a large percentage of accidents. This is illustrated by figures presented by Arnold Vey* which show that (in New Jersey) 21% of all accidents are right angle, 30% same direction, 21% opposite direction (head-on), 11% fixed object, 10% pedestrian and 7% miscellaneous. An analysis of the effect of the above listed design objectives on highway accidents may be made as follows:

If all highway and railway grade crossings are eliminated it will be physically impossible for a right angle type of accident to occur due to cross traffic and thus 21% of all accidents should be eliminated. Many types of same-direction accidents are caused by the influence of oncoming opposite-direction traffic, and if the divided highway principle is utilized, this type of causation should be eliminated. Furthermore, the use of curbs that hamper lateral freedom has an effect on this type of accident because the motorist does not have the opportunity to dodge sidewise onto the shoulder to avoid a crash in case of an

* Transactions, Vol. 104 (1939) p. 474, American Society of Civil Engineers.

emergency. Wide, smooth, stabilized shoulders, level with the pavement, permit lateral freedom and will have a favorable influence on same-direction accidents. Many same-direction accidents are caused at intersections by the stopping of vehicles at traffic lights or slowing down or stopping to make a turn. If grade separation and divided highway design with wide shoulders is utilized it is certain that same-direction accidents will be materially reduced, probably to the extent of 20% of all accidents. The opposite-direction accident can be largely eliminated by the provision of a wide space separating opposite-direction traffic, possibly to the extent of accounting for the total of 21%. If all fixed objects are removed for a reasonable distance from the pavement, the fixed-object accident can be largely prevented; but in order to be conservative it may be assumed that surely one-half of these accidents can be eliminated, namely 6%. In the case of pedestrians, if their presence is removed from the motor areas and provision is made for them elsewhere, this type of accident can be almost entirely prevented; and it is reasonably safe to state that all but 2% of the total can be eliminated, namely 8%.

To summarize, the use of known highway design technique will surely have a favorable influence on accidents and will in all probability account for the reduction of the following types and percentages of accidents: Right angle, 21%; same-direction, 20%; opposite-direction, 21%; pedestrian, 8%; fixed object, 6%; making a total of 76%. Thus, it can be seen that improved engineering technique and design will have a pronouncedly favorable effect on accidents.

The difficulty at the moment is to establish the order of magnitude of each of these features in order to adequately meet the operating needs of the motorist and at the same time remain within the realm of practical construction cost and financing. As previously noted, we are still quite ignorant of a number of these features and this ignorance should be freely confessed, for one of the surest methods

of breaking down ignorance is to become aware that there is ignorance. When this is recognized, the problem will be agressively attacked by many different minds and the problem gradually solved. Some of these thoughts have been discussed previously* but so far very little research has been initiated for the purpose of establishing definite practice. A discussion of the "factors of ignorance" may be of interest;

SEPARATION OF OPPOSITE DIRECTION TRAFFIC

The width of the median area separating opposite-direction traffic is a very unsettled matter and in many regions it has a critical relation to the economics of a highway project. In the writer's opinion, this width has a vital influence on safe speeds, for if it were infinite, head on collisions would be an absolute impossibility. Since it is impossible to construct a highway with a median strip of infinite width, the problem resolves itself into a selection of some width between infinity and zero. Possibily, some day, when all the factors are known designers will be able to write an equation, and by integrating, find the answer for each particular project. Today, so many of the factors are unknown that reduction to a formula does not appear possible; and widths must be established by cut and try analysis and economical considerations. For example, in some regions a median strip 150 feet wide may be perfectly feasible and economical while in others a very narrow width may be all that is financially possible.

After a selection is made, immediately the question arises how shall these areas be treated. Here ignorance must be confessed. It appears almost axiomatic that wide areas need no curbing, provision merely being made for hard shoulders adjacent to the pavement with grass and la ndscaping between. But as the width becomes narrower some type of control appears essential to prevent a vehicle crossing and becoming involved in head-on collisions. A statement frequently made, and one the

* Highway Research Board (1937) p. 242
* Transactions American Society of Civil Engineers (1937) p. 1018
* Journal of the Boston Society of Civil Engineers Vol. XXIV (April 1937) p. 105

writer believes very untrue, is, that it makes no difference what the width of the median strip may be as long as there is some space separating opposite direction traffic, and, that the width may be as narrow as three (3) or four (4) feet. A case in point is the accident experience on the recently opened Lincoln Highway between Irwin and Greensburg, Pennsylvania where the width of the median strip is six (6) feet.

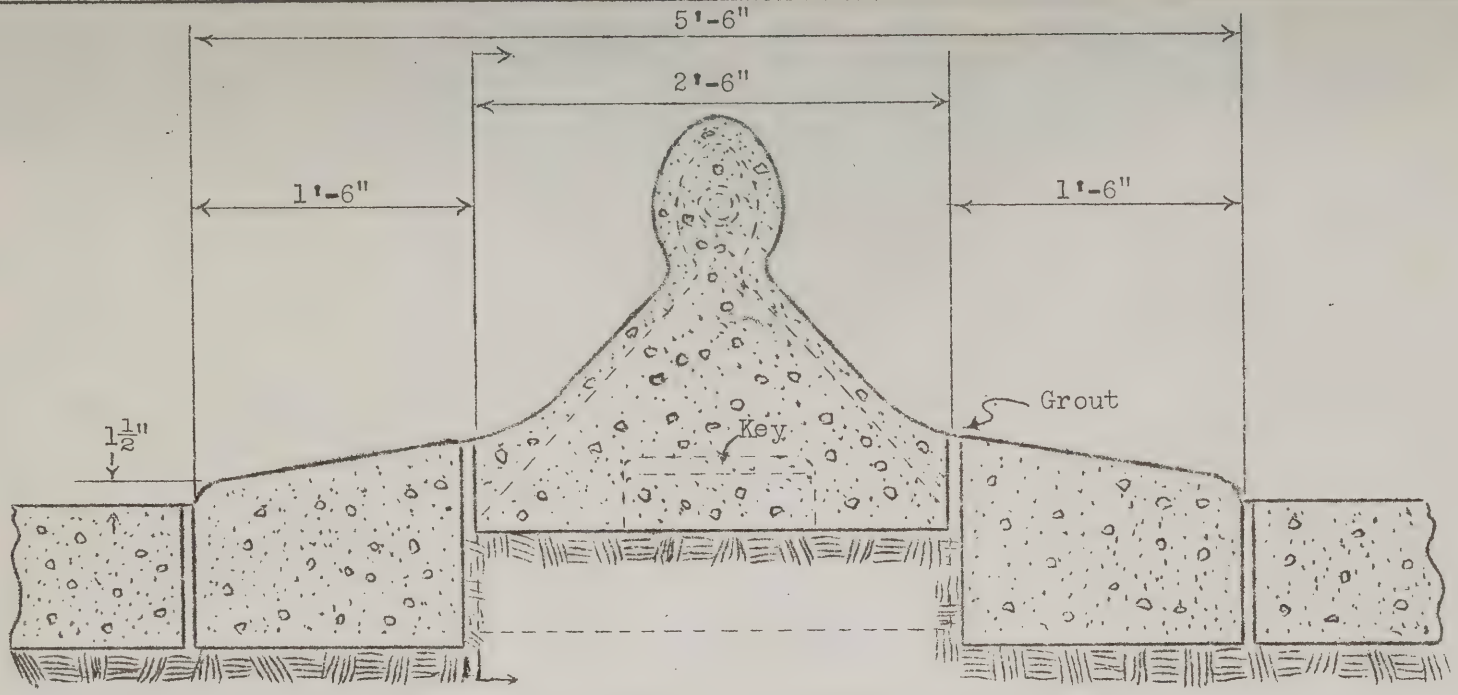
This in turn leads to the question of the safe speed to correspond with a certain median width. It is manifestly more unsafe to operate at 100 MPH when the width is only three (3) or four (4) feet wide than if the space were 150 feet wide. It would appear therefore that a classification must be made between routes located in urban areas where speeds can be reasonably rigidly controlled and where space is limited and expensive, and those routes located in rural areas where high speeds may be reasonably expected.

In urban regions, where narrow widths may be essential if financial requirements are to be met, some treatment is required to assure that vehicles will remain in their proper roadways. To date this has been met by a multitudinous variety of curbs of more or less efficiency, but no one can say, or has accumulated comparative accident statistics to enlighten the designer, as to which design is the most effective for specific conditions. It must be admitted that when curbs are utilized a certain amount of flexibility has been lost and one must expect frictions to develop, with an increase in side-swipe and rear end collision accidents. The statement has been made that where curbs are utilized a type that can be readily mounted should be adopted. This is quite true in order that a certain amount of lateral freedom may be restored yet the very possibility of mounting the curb immediately imposes another and greater hazard due to head-on collisions. Having reached this impasse, the only logical design where narrow median widths are necessary

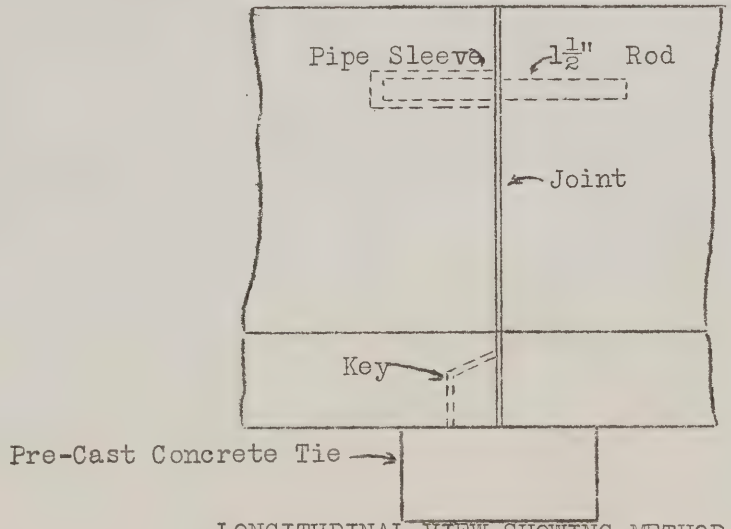
appears to be a continuous barrier which will physically prevent the encroachment of traffic in opposite direction roadways. A suggested type is illustrated in Figure 1 which utilizes a design of separator developed experimentally by the Bureau, Michigan Highway Department and General Motors Proving Ground. Precast concrete is utilized instead of steel in an effort to provide a more economical design. The sloping curbs are provided to give the motorist a "second chance" by allowing additional elbow room beyond the normal paved lane. With this type, overhead bridges for grade separation must have a clear span across the entire graded roadway with no center piers, unless the roadways are splayed out, because there is not sufficient width for a center pier and the use of a center pier would create a definite hazard. The estimated cost of the separator and curbs is $4.50 per linear foot, or approximately $24,000.00 per mile.

It would be possible to extend the discussion on median strips in urban areas but the primary purpose here is to present problems relating to the express highway in rural areas and to illustrate the discussion with information and experience gained during the construction of the Pennsylvania Turnpike.

Last year Mr. Halsey posed the question as to whether it would not be better in rural areas where excavation was heavy to utilize the same narrow median width and separator illustrated in Figure 1 in order to effect a saving in excavation costs, while at the same time provide inherent safety to the motorist. This is a very pertinent question and the writer has been thinking over every phase of the problem during the past year not only from the standpoint of motor safety and speeds, but also from the intensely practical viewpoint of economics. Frankly, the writer is a proponent of the wide median strip but nevertheless must admit there are regions and conditions which require a modification in order that projects may remain in the realm of financial possibility.



SECTIONAL VIEW OF NARROW MEDIAN STRIP



LONGITUDINAL VIEW SHOWING METHOD OF KEYING ADJACENT SECTIONS TOGETHER

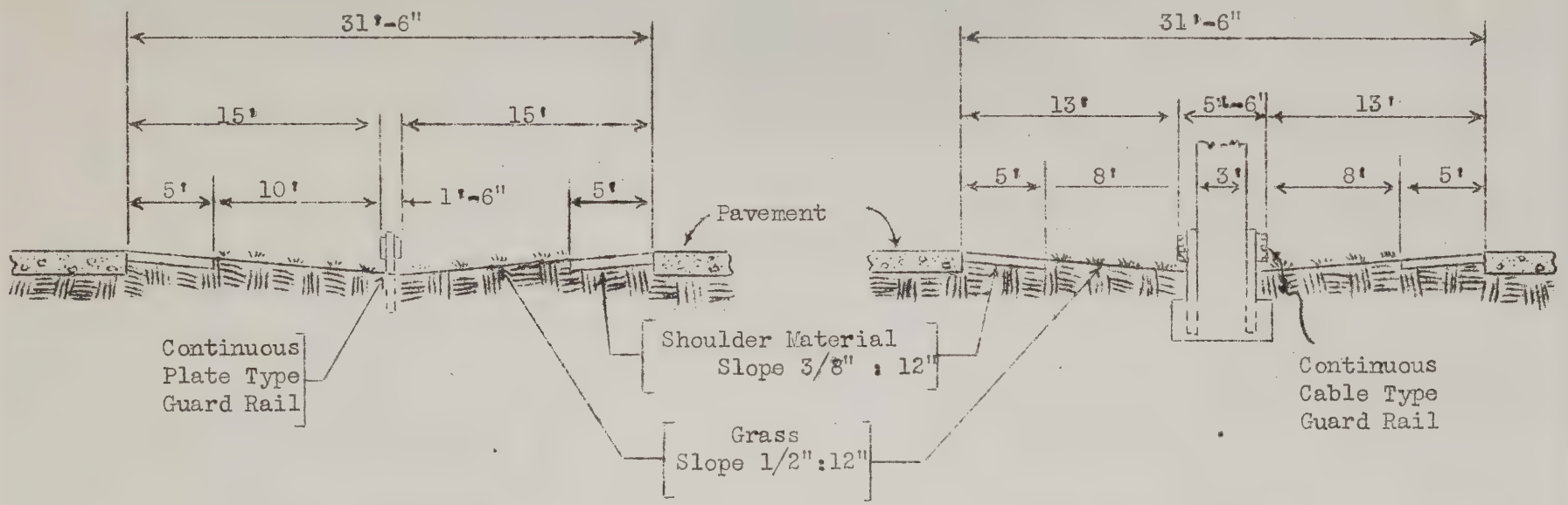
FIG. 1

It does not appear to the writer that the narrow median width illustrated in Figure 1 is suitable for speeds in excess of 50 MPH because of the limited amount of "elbow room" adjacent to the inside lanes and a study has been made in an effort to develop a minimum design suitable for high speeds. Because of the narrow width it is considered essential to provide a continuous separator and since, in the design under consideration, economy is paramount it is necessary to develop an inexpensive type which will at the same time be effective.

The following is submitted as a minimum for rural areas where speeds in excess of 50 MPH are expected. The design, Figure 2, is based on providing a "factor of safety" space of 15 feet adjacent to the inside or "fast" lanes and an inexpensive type separator in the center to prevent "out of control" vehicles from encroaching within the opposite direction roadway. The separator is a simple guard rail type with a spring steel "can" attached on each side of the post and cables or steel plate strung along the outside face of each "can". It is estimated that this separator will cost $1.50 per linear foot in place, or approximately $8,000.00 per mile.

The hard shoulder, 5 feet wide, adjacent to the pavement is to be noted. Observations along existing divided highways furnish mute evidence that a hard surface adjacent to the inner lanes of pavement is a necessity if safety is to be preserved.

With the type of design shown in Figure 2 it may be permissible to use a center pier for overhead bridge crossings. The lower view in the illustration shows a condition with a center pier where it can be seen that a width of only 2 feet on each side has been lost due to the influence of the center pier. It may be further noted that the guard rail posts are carried straight through and this was shown purposely because it is felt that a "cushioning" effect should be placed around the pier, which may be considered an "immovable" object. These views show what, in the



CENTER MEDIAN ZONE

MINIMUM DESIGN

IN THE OPEN

CENTER MEDIAN ZONE

MINIMUM DESIGN

AT BRIDGE PIER

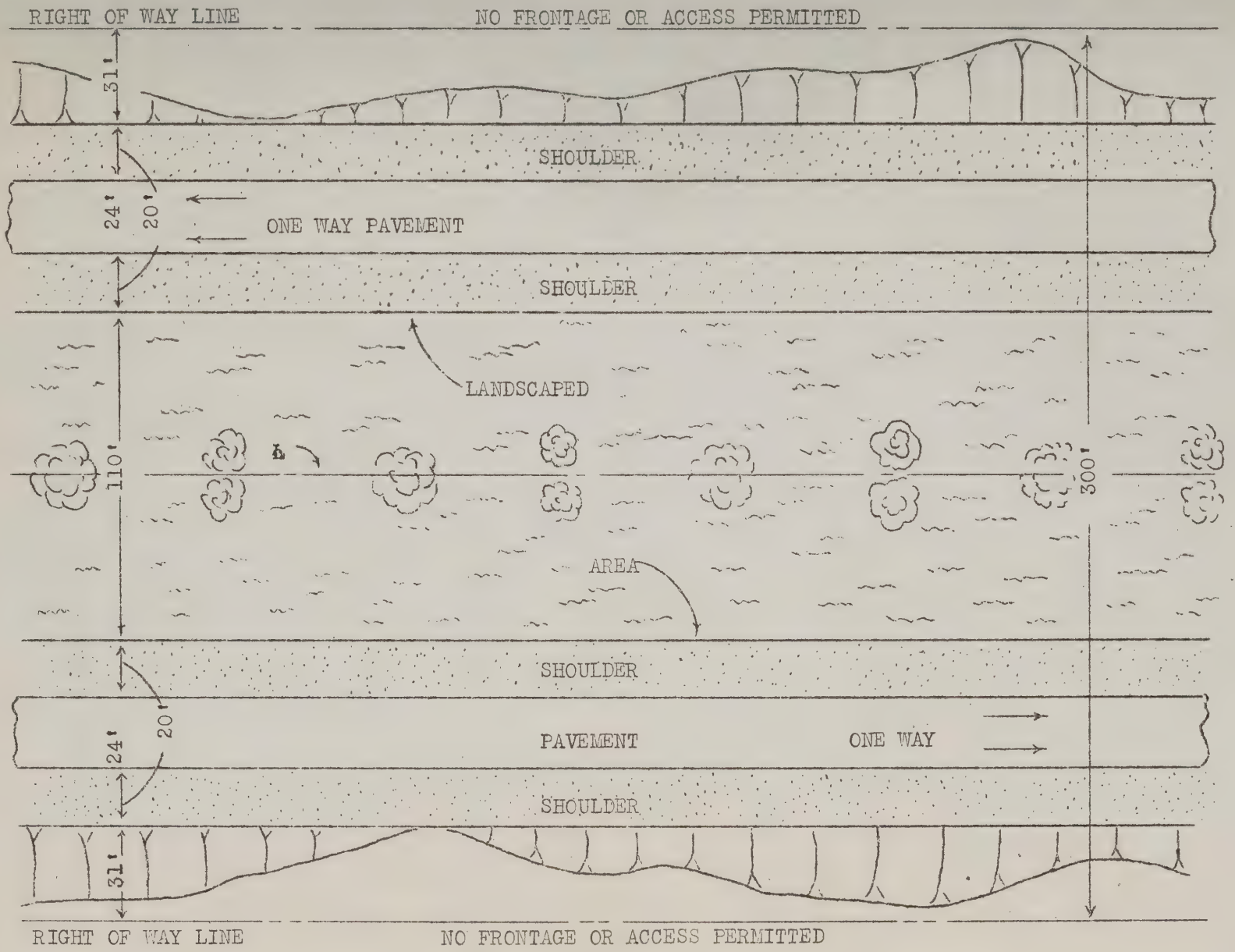
MEDIAN ZONE

FIG. 2

writer's opinion, should constitute the minimum width for use on a limited way type of express highway for all types of topography except that classified as "mountainous" and where it is intended to use the "barrier" type of design. To date no data has been accumulated, or is there any authoritative information available, from which may be deduced the minimum safe width when no barrier is utilized to prevent the encroachment of opposite direction traffic.

In Figure 3 is illustrated a design which may be considered the ideal for high speed operation and it is hoped that this design will be utilized wherever practicable.

Inasmuch as this is a practical world where "money talks" it will often be necessary to adjust the width of the median strip so that excavation quantities will not exceed a given amount of excavation in order to confine the cost of a project within a predetermined sum of money. This has frequently happened in the past and should serve as a caution to engineers not to prepare preliminary estimates for this type of construction on too low a basis. Nevertheless, this situation must often be faced and it will be necessary to make an economic comparison between the wide median areas without a barrier and a design having a narrow median strip with a barrier. For example, if we use the quantities of excavation for the Turnpike (this territory would be classified as "mountainous") we find that the average depth of excavation for the entire distance (assuming no fills) amounts to 8.6 feet. The guard rail type of barrier shown in Figure 2 is estimated to cost $1.50 and consequently the width of the median strip could only be increased approximately 11 feet before the cost of a barrier was equalled (using Turnpike Unit prices). It can therefore be seen that in this type of country it would not be economic to provide an "open" design. In easier country, however, particularly in the coastal plain and prairie areas, it may be quite as economic to use an "open" type design instead



EXPRESS HIGHWAY WITHOUT MEDIAN DIVIDER

FIG. 5

of the "barrier" type, because in such country it will be unnecessary in many cases to perform any grading within the median area except for the shoulders adjacent to the pavement and the cost would be limited to right-of-way, clearing of trees and obstructions which, as you will note subsequently, are not serious items of cost in the average rural area. Even when it is desired to grade the entire median area, if the average depth were one foot (continuous) the width could be increased 93 feet before the cost of the barrier is equalled, thus making a total median width of 124½ feet. It is also to be noted that the classification of excavation material encountered will affect the unit price and in easy country the price should be lower than on the Turnpike where a large percentage was of rock. In estimating, caution must be exercised when preparing comparative figures for wide roadways in sidehill territory.

As a matter of further interest it may be noted that the median width under Turnpike conditions could be widened 32½ feet (making a total width of 32½' + 5½' = 38 feet) before equalling the cost of the separator shown in Figure 1. This width is still too narrow to eliminate a separator but in all probability the type shown in Figure 2 could be utilized, in which case the roadway could be widened 21½ feet (making a total width of 27 feet) and a Figure 2 separator utilized before equaling the cost of the separator in Figure 1. For Turnpike conditions a border line case is encountered, it being slightly cheaper to use the separator shown in Figure 1 than the construction shown in Figure 2, the width falling short 3½ feet.

It is to be specifically noted that although there has been considerable discussion with respect to the width of median strips, no serious attempt has ever been made to discover the relation between width and safe speed for vehicle operation and this is a definite challenge to the engineering profession.

ELIMINATION OF GRADE CROSSINGS

As noted previously the intersectional or right angle type of accident accounts for 21% of all accidents and the complete elimination of all grade crossings

and local access should largely eliminate this type of accident. In addition to the safety factors, the elimination of cross traffic, signalized intersections and the avoidance of towns has a vital influence on the time required to cover distance by a motor vehicle. It is this time element which is of intense interest to the average motorist. In many sections it is often impossible to average more than 30 MPH, and this very fact tends to intensify the accident situation because after being held up by signal lights and cross traffic the driver often speeds up to a dangerous rate not warranted by the actual operating conditions in an effort to make up for lost time. With all grades separated and the elimination of all "surprise" local entrance points it will be possible for the motorist to "set" his speedometer at the rate which he desires to travel and continue uniformly at that rate to the end of the journey provided the alignment is so designed. Under these conditions it will be possible to accomplish an average speed within two or three miles of the speedometer reading. That these savings are material is indicated by the following tabulation which was computed on the basis of the length of the Pennsylvania Turnpike:

AVERAGE SPEED M. P. H.	HOURS TO DRIVE TURNPIKE	SAVING IN MINUTES PER 10 MPH INCREASE IN SPEED
30	5.33	80.0
40	4.00	48.0
50	3.20	31.8
60	2.67	22.8
70	2.29	17.4
80	2.00	13.8
90	1.77	10.2
100	1.60	

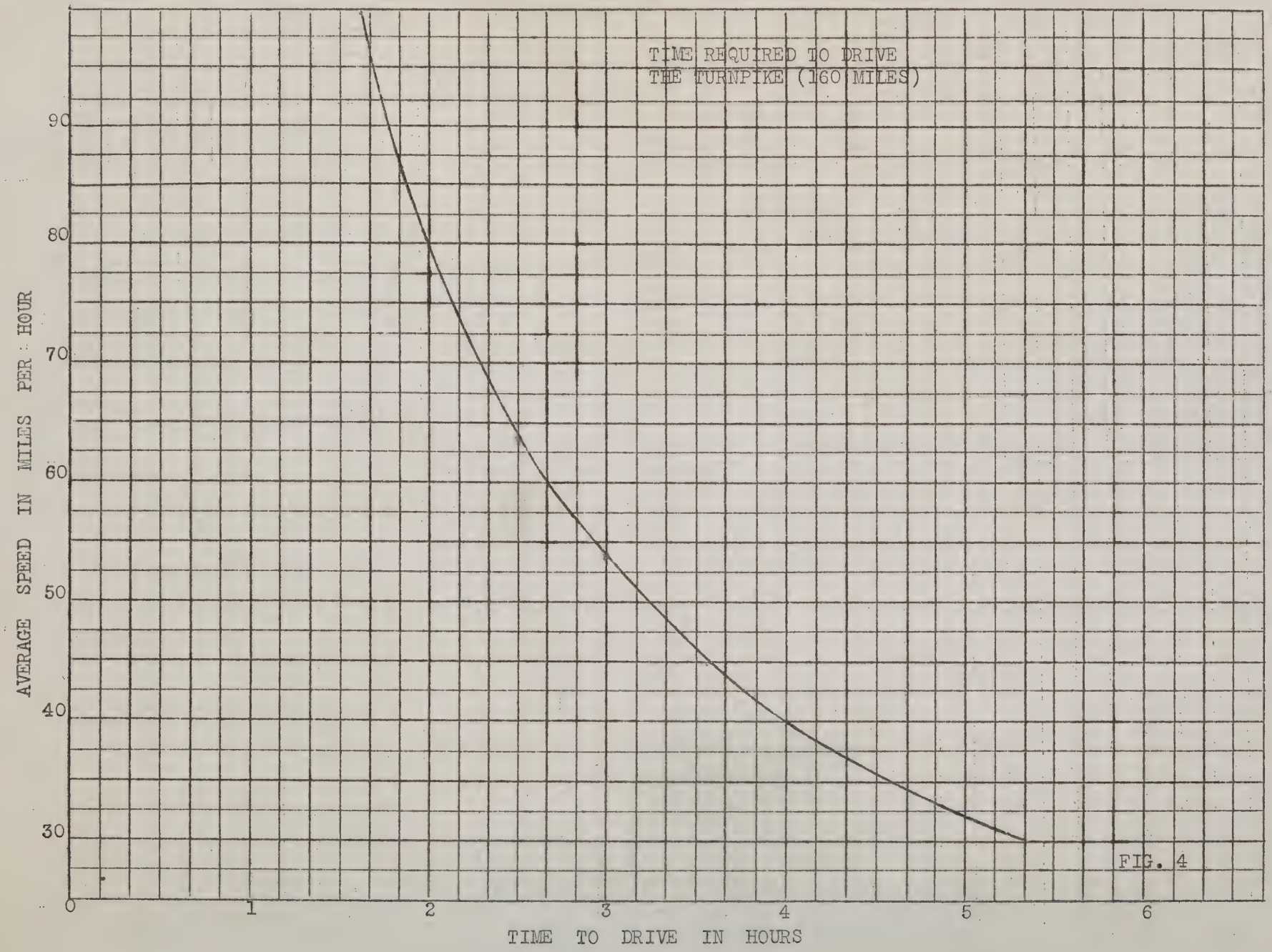

TIME REQUIRED TO DRIVE
THE TURNPIKE (160 MILES)
AVERAGE SPEED IN MILES PER HOUR
90
80
70
60
50
40
30
0
1
2
3
4
5
6
TIME TO DRIVE IN HOURS

FIG. 4

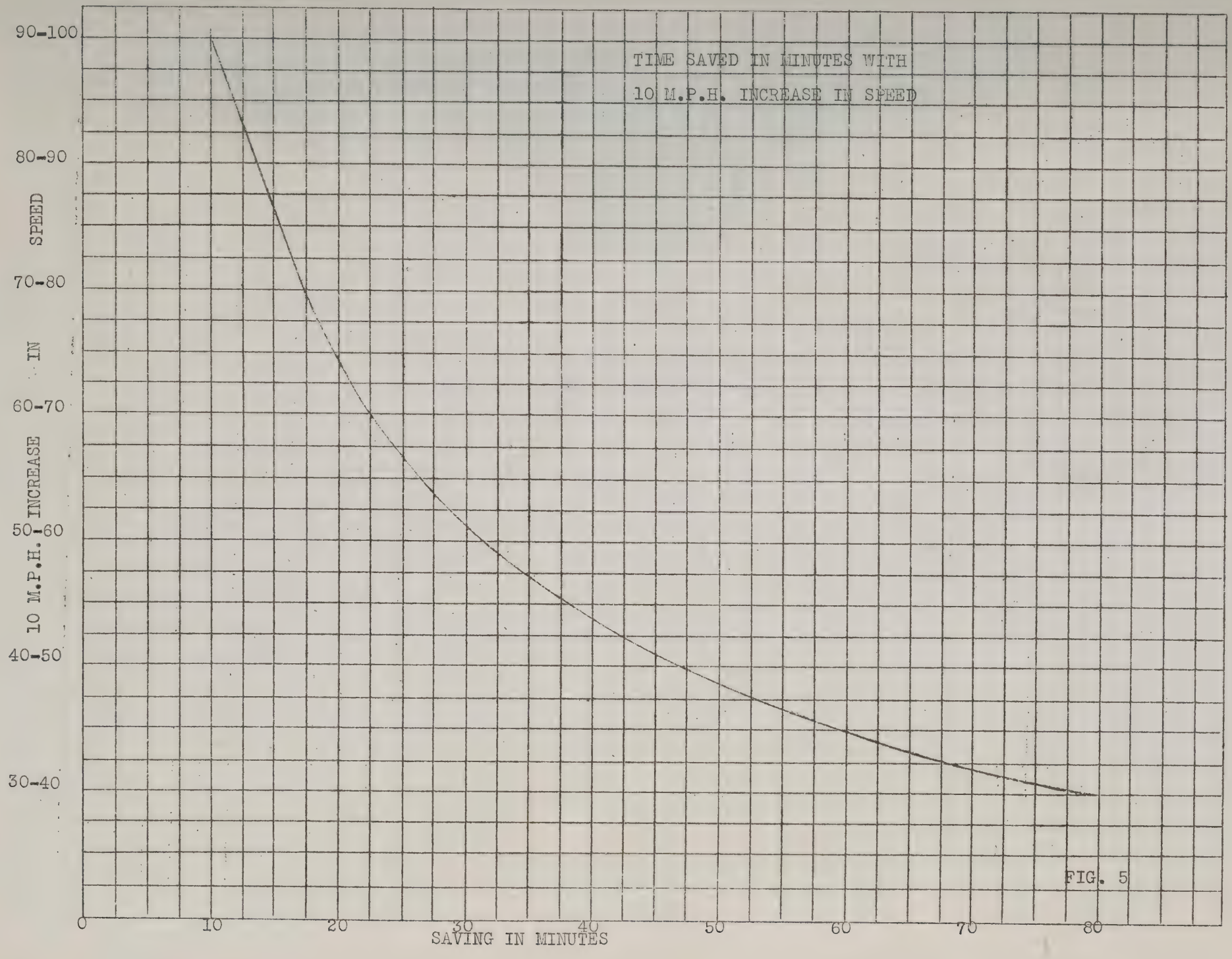

TIME SAVED IN MINUTES WITH
10 M.P.H. INCREASE IN SPEED
90-100
80-90
70-80
60-70
50-60
40-50
30-40
SPEED
IN
10 M.P.H. INCREASE
0
10
20
30
40
50
60
70
80
SAVING IN MINUTES

FIG. 5

The above data is graphically illustrated in Figures 4 and 5.

At the present time there is no breakdown of figures which will indicate the net cost of grade separation on the Turnpike, but an analysis of the figures that are available indicates that the cost of grade separation, including highway relocation and restoration, does not exceed 9% of the total cost of the Turnpike. Consulting the above tabulation and making the necessary computations, the savings realized by various assumed increases in average speed due to grade separation expressed in percentage are as follows:

An increase in average speed from 30 to 50 MPH results in saving 40% in time.
An increase in average speed from 40 to 60 MPH results in saving 33⅓ in time.
An increase in average speed from 50 to 70 MPH results in saving 28% in time.
An increase in average speed from 60 to 80 MPH results in saving 25% in time.
An increase in average speed from 70 to 90 MPH results in saving 23% in time.
An increase in average speed from 80 to100 MPH results in saving 20% in time.

It is also interesting to note that a saving in time of 16⅔% will result from such a moderate increase in speed as from 50 MPH to 60MPH and it will be noted that this saving is almost double the percentage of increase in cost. In other words, in this latter instance, the returns in time saving to the motorist is almost double the cost of the features which will make this time saving possible.

In order to illustrate the economic value due alone to time savings, a computation based on the volume predicted for the first year (1,300,000 vehicles,) in the Traffic Engineer's report on the Turnpike, assuming the worth of passenger car time at 30 cents per hour and truck time at $3.00 per hour and calculating the cost of stopping a vehicle from Wiley's "Cost of Passenger Car Stops"*, showed the annual benefits to amount to $609,300.00. No credit for economic saving due to accident reduction because of grade separation was included in this figure. The full annual cost, including interest at 3-3/4% on the total investment, amortization in 30 years

*Civil Engineering, May 1935.

and maintenance, amounts to $284,500.00. Therefore the actual annual benefits are computed to be more than double the annual cost.

In computing the time savings, the conservative assumption was made that average passenger car time would be increased from 50 MPH to 60 MPH and the number of stops and time losses suffered by trucks was estimated from known conditions along the Lincoln Highway, but no account was taken of time savings due to the light grades on the Turnpike. It is to be noted that the benefits of $609,300.00 realized during the first year will continue to increase during each year for the entire life of the project while the annual cost of $284,500.00 will remain constant for 30 years when it will automatically be liquidated in full.

It would therefore appear that where any appreciable volume of traffic is involved the cost of grade separation is economically justified from the standpoint of time savings alone, the additional economic savings due to accident reduction and the suffering and sorrow occasioned by traffic accidents being so much extra in dividends. The present method of designing highways, loaded with grade crossings, for speeds of 50 or 60 MPH and then routing them through the center of every town does not appear economic when the average speed may be reduced to 30 MPH because of lack of grade separation. In other words after investing considerable sums of money to achieve a 60 MPH highway such investment is largely rendered ineffective insofar as a reasonable average speed is concerned because the grades were not separated. This is particularly noticeable in the lower speed brackets, for it is here that the greatest possible percentage in time savings are realized due to the provision of grade separation.

In connection with grade separation it may be pertinent to discuss the width of bridges in general, inasmuch as highway and bridge engineers always display an attitude of dismay at the cost of bridge construction and have, therefore, attempted

to reduce this cost by the doubtful expedient of providing bridges of less width than the graded roadway. This policy was resulted in economic costs from two causes, (a) accident losses, and (b) loss due to reconstruction of bridges when the narrow width becomes obvious and when widening of highway is necessary. There is a point, however, as the length of the bridge increases, where it will be necessary to consider the possibility of narrowing the structure to some width less than the graded width of the highway in order that proportional cost may not get too badly out of line. At the moment, there is no data available to form a basis for judgment as to limiting lengths and widths. In other words, there is no concerted opinion as to whether the limiting length of a bridge should be 500 feet, 1000 feet or 2000 feet where a reduction in width is justified and to what extent a structure should be narrowed in relation to the width of the normal median strip and the normal shoulders. From the standpoint of the motorist, who will flash by a 1000 foot structure in less than 10 seconds at 70 MPH, width reductions for a structure of even that length do not appear justified and the motorist particularly cannot understand why there is any justification whatever for narrowing structures 100 feet or less in length when he normally flashes by them in a fraction of a second. In railway practice it has been possible to issue and enforce "slow orders" at weak and dangerous bridges, but such "slow orders" have not been very effective in dealing with the American motoring public.

In the past there has been a regrettable lack of streamlined railings and parapets whether of full width or narrowed construction. There has been a general disregard of continuity between guard railing and parapet walls and a very "ragged" appearance and dangerous condition from an accident standpoint results where the guard rail ends and the bridge parapet wall starts. It must be remembered that guard rail is somewhat flexible and will, when being struck by a vehicle, yield a

certain amount thereby cushioning the impact effect, while on the other hand, a fixed parapet wall presents a condition which may be considered an "immovable" object. In smaller spans there appears merit in the possibility of carrying guard rail type of construction straight across the bridge so that complete continuity is secured as well as presenting a uniform appearance to the motorist. It would, of course, be necessary to provide sufficient strength in the guard rail structure at such crossings so that vehicles could not break through and fall onto the lower roadway. It is difficult for the motorist to understand why guard rail is stopped and a concrete railing structure erected for a length of 20 or 30 feet and it must be admitted that this type of construction has been prompted by a desire to present a pleasing appearance to persons standing outside the main roadway. However, it must be remembered that in rural districts, where for every person who observes the aesthetic effect of a structure from the outside there will be thousands who will observe the effect while traveling along the main roadway and, consequently, the aesthetics should cater to the greater number.

The use of center piers has become one of the most controversial issues in the highway engineering field, but unfortunately the issues have not been clarified and there has been no authoritative definition of policy with respect to their use. Anyone familiar with structures instantly realize that the use of center piers will decrease the cost of construction, not only because of the reduction in span length but also because the shallower thickness of the structure decreases the amount of rise on the adjacent highway. There is thus a powerful incentive to use center pier construction without regard to accident possibilities and without provision for the protection of the motorist from the hazards presented by the obstruction. Consequently, center pier construction should not be utilized unless the median strip is of suficient width; and protective devices constructed

in such a manner that the "fixed object" type of accident is replaced by a "sliding" type accident. In the design of such protective devices recognization should be given to the fact that concrete piers are of the "immovable object" type, and it is essential that such piers be screened with protective devices which are resilient in order to provide a cushioning effect against impact forces.

In the construction of a grade separated type of highway it is often possible to reduce the costs of grade separation by relocation and reconnection of existing highways, particularly those of a low type character. Although the desirability of such relocation may be obvious to everyone concerned, the consummation may be impossible of fulfillment because of present laws respecting highways in many states. This is illustrated in the State of Pennsylvania where the law specifically provides that the state highway cannot be relocated unless such relocation results in (a) shortening of distance, (b) improvement in safety, and (c) a reduction in maintenance cost.

The relocation of township (mud) roads presents an even more serious difficulty because it is not possible to relocate such roads, even though the traffic may amount to only one vehicle per day, without first obtaining the written consent of the township supervisors and, in addition, each person residing along the road. On the Pennsylvania Turnpike it has actually been necessary, because of the inability to secure the necessary consents, to construct grade separated structures, which together with the necessary approach grading, cost from $25,000 to $35,000 each, on mud roads carrying a volume of traffic amounting to two or three vehicles per day.

It must be admitted that the purpose and necessity of these laws has been justified, in that they protect the public against unwarranted exploitation by railroads and other private organizations whose motives are impelled by selfish interest. With the advent of the limited way express highway, which will in all

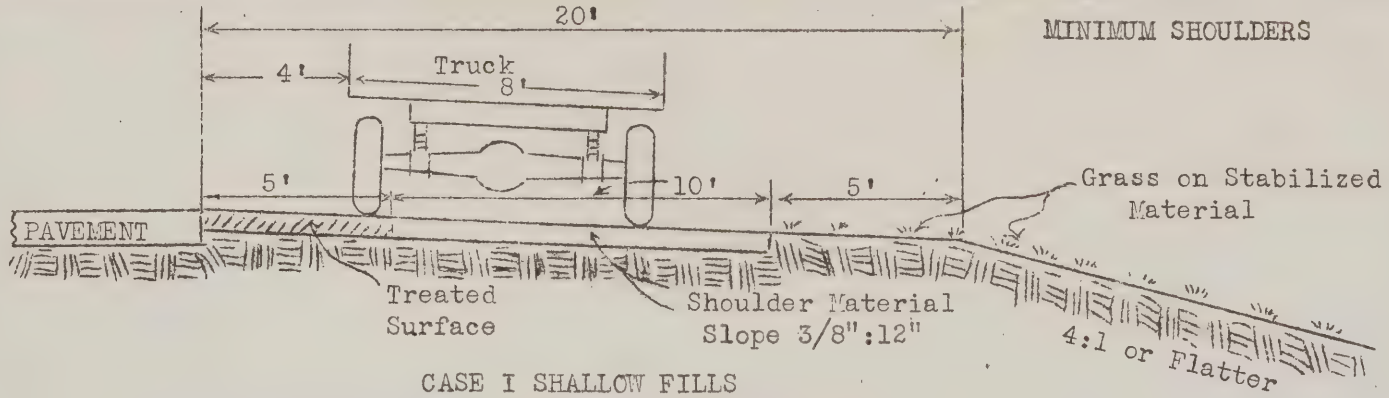
probability always be constructed by a public body, it would appear equitable for the laws to be amended in order that grade separation may be based on over all economics and the greatest good for the greatest number of people.

At the moment, to the writer's knowledge, no movement has been started to correct this serious situation in any of the states and there is opportunity for constructive action in this field. The first step would be for responsible persons in each state to study the present laws and devise equitable amendments for submission to the legislatures. The next and most difficult step will be the promotion of such legislation until it has been successfully passed and becomes law. The seriousness of the situation can be visualized when it is realized that tens of millions of dollars are involved and the fact that the economic justification for certain express routes may hinge on sufficient reductions in grade separation construction costs to bring the project within the realm of economic justification.

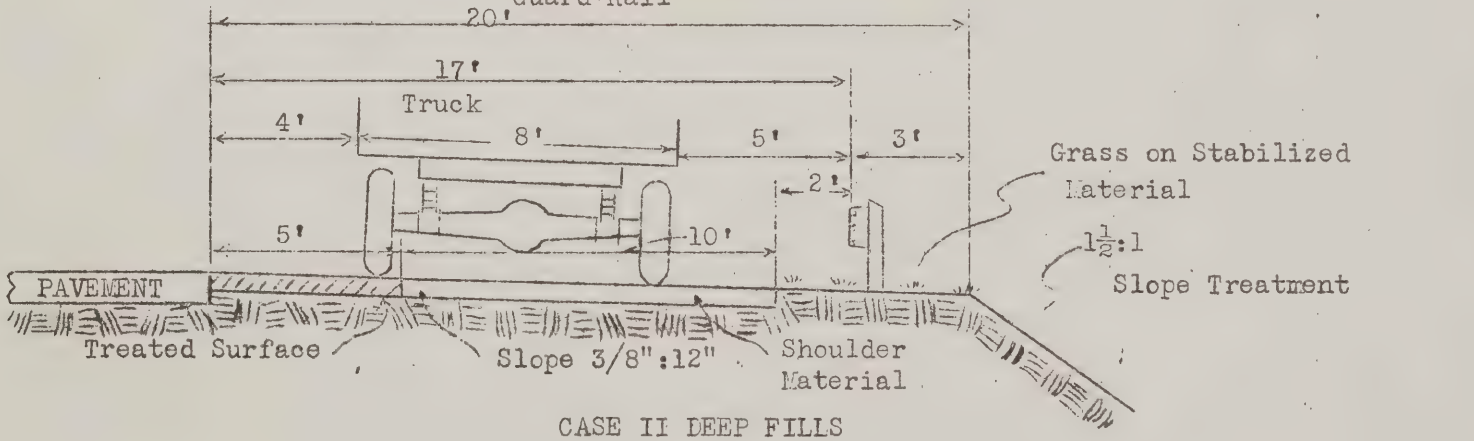
SHOULDERS

There appears to be confusion in the minds of many engineers as to the function and purpose of the shoulder alongside the paved area of a highway. The real purpose is twofold, (1) a space for stopped vehicles to park clear of the traveled roadway, and (2) a "factor of safety" space providing lateral "elbow room" for cars to dodge out onto in case of a traffic emergency or on which to recover from a minor mishap before it can develop into a major disaster.

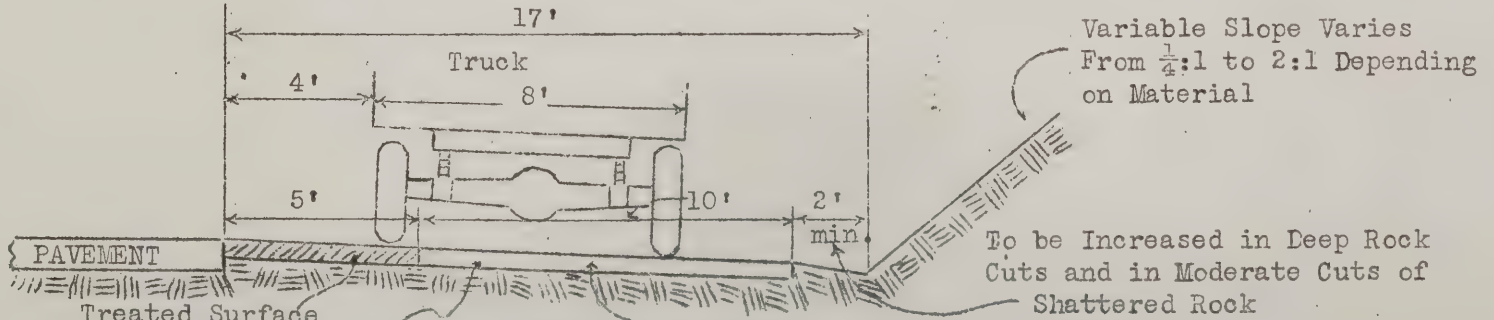
The most question awaiting an answer is, just what the width should be to meet these two objectives: For the former, it is possible to arrive at minimum dimensions by analyzing the space requirements by a comparison with the actual width of trucking units. This comparison is shown in Figure 6 in which three cases are illustrated; fills of moderate depth, deep fills requiring guard rail and cuts. Inasmuch as shallow fills do not present a problem from an economic standpoint, the width of Case I is shown the same as Case II for the sake of uniformity as well as to provide more space for the vehicle because there is no barrier to prevent an



CASE I SHALLOW FILLS
Fill of Moderate Depth with Flat Side Slopes No Guard-Rail



CASE II DEEP FILLS
Deep Fill with Guard Rail



CASE III CUTS

FIG. 6

"out of control" vehicle from leaving the graded roadway. In considering Case 11, it is thought highly dangerous for a vehicle to be parked closer than 4 feet from the edge of a high speed lane of pavement particularly when it is considered that the truck attendant may be standing or working alongside. In order to assure that the average vehicle operator will park this far clear of the pavement it is necessary to consider the psychology of the driver and to realize that he will not park clear unless there is ample room to his right or "off side". The driver cannot see the right fender too well and is inclined to allow ample room on that side when parking next to an obstruction such as a guard rail. Furthermore, a tire replacement or inspection may be necessary and a driver will make sure that there is plenty of working space left so that he will not be inconvenienced or hampered while performing the repair. The figure indicates a width of Stable Shoulder on which the operator can work clear of grass, of 15 feet, which provides a clearance next the pavement of 4 feet and on the "off side" of 3 feet. It is most desirable from a safety standpoint for the face of the guard rail to be set clear of the stable shoulder by at least 2 feet and in order to assure the stability of the guard rail on deep fills, as well as to allow for the inevitable sloughing or rounding at the edge, an additional 3 feet is desirable from the face of the rail to the break of the shoulder at the top of the fill. This reasoning results in a total width of 20 feet. In the case of cuts, because of the absence of railing and the necessity for its support at the top of a fill, a width of 17 feet is indicated.

On studying the views it will be noted that the 5 foot width immediately adjacent to the pavement is shown with a type of surface composed of inexpensive broken stone bitumin treated, and this serves the important purpose of providing a safe and reliable surface onto which a vehicle may pull unhesitatingly at high speed. There is nothing more dangerous than to ride off the edge of a pavement at high speed onto a sloppy unstable quagmire into which the wheels of the vehicle will sink several inches. This condition has been responsible for many disasterous accidents. The

remainder of the usuable shoulder is shown to be provided with stablized material which will, under all weather conditions, maintain a solid rutless surface and which can be maintained by a blade machine. The last 5 feet of shoulder is shown in grass in order to present a color contrast and to protect the edge from washing and erosion.

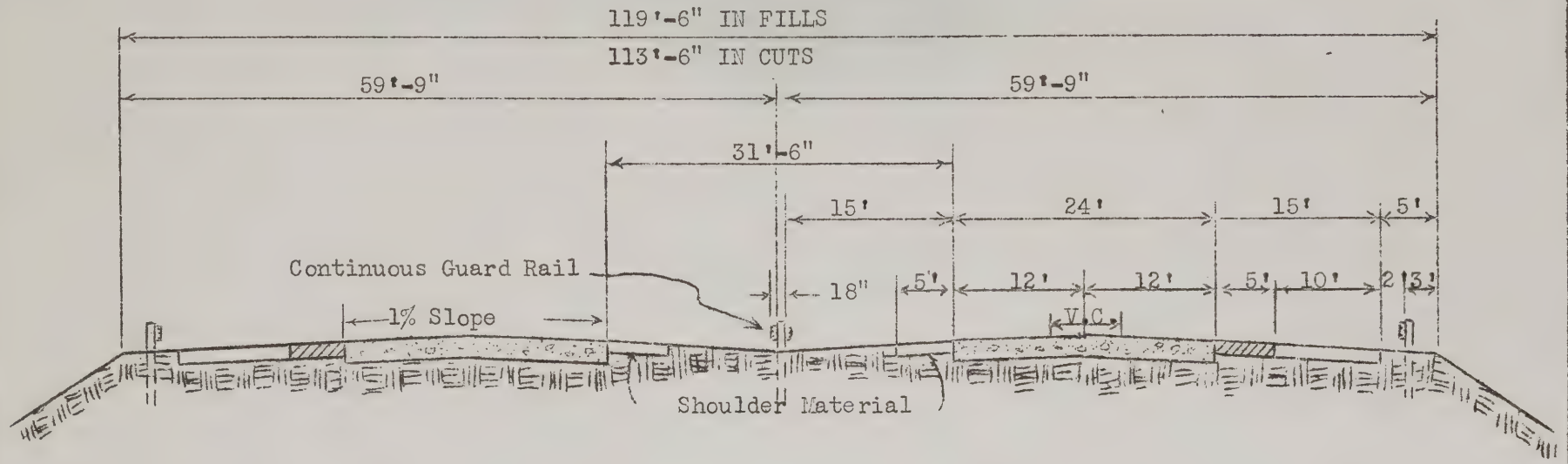
Particular attention is directed to the flat fill slope indicated in the top view of Figure 6. Such easy flowing cross profiles with the edges rounded are most desirable not only because of the greatly increased safety presented for vehicle operation, but also because of the pleasing aesthetic effect produced. This is particularly true where soil conditions or topography require open ditches to lower the ground water table. Any depth ditch may be obtained by the simple process of continuing a flat slope from the shoulder until the desired depth is reached. It should be noted that the back slope of such ditch should also be flat, preferably not steeper than 2:1, and all edges rounded in order to produce a flowing cross profile. Where this type of construction is utilized caution must be exercised that right-of-way of sufficient width is obtained.

The second objective in the determination of shoulder width, the "factor of safety" space* is much more difficult of determination and it is here that fundmental research is required in order that well substantiated facts may be made available so that a balance may be secured between safety requirements and economic considerations.

Combining into Figure 7 the data shown in Figures 2 and 6 results in a suggested minimum design for express highways designed to carry traffic at speeds in excess of 50 MPH.

It will be noted that a two way crown is shown for each roadway. This will tend to increase the cost due to the additional center drainage required but it is felt that the advantages afforded in less water on the pavement, freedom from

* Highway Research Board (1937) p. 246



SUGGESTED

MINIMUM CROSS SECTION

FOR

EXPRESS HIGHWAY

FIG. 7

water from windrows of melting snow and betterment in superelevation runout at the ends of spirals more than justifies the additional expense. Center drainage inlets should be set nearly flush with the surface and placed in the center of the median strip. Observation also indicates that a two way crown tends to discourage motorists from riding the inside lane, except when necessary to perform a passing operation. Narrow width median strips and curbed designs do not lend themselves to two way crown construction and in these cases the slope should be continuous each way from the center line of the median strip.

PAVEMENT

The cost of the pavement constitutes the greatest single item of capital expenditure in the cost of a completed highway project. It is therefore essential that it be placed on alignment, grades and right-of-way width which will not become obsolete before the end of its useful life, otherwise a serious economic loss in capital investment will result.

The utilization of the divided highway principle requires the use of two lanes of pavement in each direction. In addition to the inherent safety features provided by this principle of design, facility of traffic movement is greatly increased because passing slowly moving vehicles may be accomplished without delay, and this feature has a direct appeal to the motoring public. Where financial limitations prevent the initial construction of four lanes of pavement, a two lane roadway may be provided on one side of the right of way in the final position of the ultimate improvement and this insures the project against a serious financial loss in the future when the volume, speed and importance of the traffic will require four lanes.

The present trend rather definitely indicates a width of 12 feet for lanes where high speed may be expected and where no curbing is utilized. When curbing is utilized wider lanes are required because of their effect on restricting "elbow room".

*Highway Research Board (1937) p. 246

One of the most effective safety measures as well as the most efficient time saving devices is the restriction of the right of access and frontage coupled with the elimination of all cross traffic. Many modern highways, particularly those carrying a large volume of traffic, have been throttled, their economic transportation efficiency vitally reduced and the hazards of operation increased by residential and commercial development. Rural farm entry, although not as serious from the standpoint of lowered transportation efficiency, nevertheless presents a serious accident hazard on high speed routes because the entry of vehicles is uncontrolled and often takes place unexpectedly without adequate warning.

It is somewhat unfortunate when a highway is constructed with funds contributed by the user, for adjacent property owners to reap a substantial profit at the expense and to the detriment of the persons (the motorists) who furnished the money, through increased land values brought about by the improvement. This is particularly unfair because the beneficiaries rarely bare any of the direct expense and procure such benefits merely because they are, by an accident of fate, located along the route of the improvement.

Unfortunately it is difficult to remedy this situation without placing freeway legislation on the Statute books of each state. There has been some progress in solving the problem but it is most imperative that immediate steps be taken to obtain carefully drawn legislation in each State in order that the tremendous economic waste now occurring due to unrestrained right of entry and frontage may be checked. Here is an opportunity for constructive work.

In view of the increasing seriousness in the pedestrian type of accident on rural highways it is essential that their presence be removed from express highways, where because of an anticipated increase in speed the problem will increasingly become aggrevated. Here again legislation is urgently required to make it unlawful, punishable by fine and imprisonment, for pedestrians to trespass within the right of way of express highways. The hitch hiking nuisance and menace is only one phase

of this problem.

DESIGN SPEED

To the writers knowledge, no real definition and criteria as to what constitutes design speed has been set up, yet this factor is a fundmental which vitally affects every phase of the design as well as the foundation of highway economics. When a design speed has been selected it immediately becomes possible to think of the forces and loads to be carried by the highway in terms of vehicle operation in seconds of time and to correlate observed human reaction with vehicle velocity.

Although nothing has been crystallized, there are apparently two schools of thought, one promoting the principle that the design speed should be considered the maximum speed likely to be attained by the motorist driving a modern car, with some allowance for additional speed because of the continuing development of the vehicle, as for example 100 M.P.H., while the other school contends that the design speed should be set at a more moderate rate, as for example 70 M.P.H., but that the design should be checked or tested to make sure that disaster will not automatically ensue at a higher rate known on the "critical" speed. To date, among proponants of both schools there has been a regrettable tendency to assign values, such as friction factors, that were not reasonably conservative. This tendency is understandable when the economic effect on the completed design is considered and when it is realized that there is an almost complete lack of test data for operation at speeds above 70 M.P. H. Nevertheless the fact remains that many designs are unbalanced in that there are weak links in the chain while other elements are overdesigned. Good engineering as well as sound economics dictates that it is just as important for the various elements of a highway to be properly proportioned as it is for the members of a bridge. Weak links (members) are not tolerated in bridge design and they should not be in highway design. It is therefore of the utmost importance that a sound and uniform policy be established prior to the initiation of any considerable program of express highway construction; and it is hoped that the problem will be attacked with vigor

and intelligence.

Because of the uncertainty surrounding the subject of design speed it may prove of interest to outline the specific elements which are effected by design speed and to discuss briefly what is known and what is not known about each. Design elements affected by design speed are as follows:

DESIGN ELEMENTS AFFECTED BY DESIGN SPEED

(1) Overall width of highway

  (a) median strip, width and treatment.

  (b) shoulders, width and treatment.

  (c) paved lane width.

(2) Sight distance

  (a) Vertically

  (b) Horizontally

  (c) Determination of proper friction factor and reaction time.

(3) Minimum radius for horizontal curvature.

(4) Rates of superelevation for horizontal curves.

  (a) Determination of proper friction factor.

  (b) Correlation of rate with requirements of slow and fast vehicles

(5) Length of Spiral Curves.

(6) Length of superelevation runout beyond spiral

(7) Length of profile tangents

(8) Distance between horizontal curves.

  (a) Same direction

  (b) Reverse direction

(9) Spacing of points of access and exit.

(10) Length of acceleration and deceleration lanes at points of access and exit.

(11) Design of turnouts at points of exit

(12) Signs

  (a) Size of letters

(b) Maximum number of words in sign message

(c) Reflectorization

(d) Position

(13) Type of pavement.

The overall width of the highway has been previously treated.

SIGHT DISTANCE

Inasmuch as express highways of the divided type are being considered herein it is unnecessary to discuss the sight distance required for passing operations and consequently only non passing sight distance will be treated. This sight distance may be known as Rear End Collision Sight Distance and has been defined as the distance required, at the design speed, to bring a car to a stop in time to avert a collision in the event there is an obstruction in the highway. This in turn depends on an interval of time required for perception and brake reaction, plus the distance necessary to bring the car to a stop after the brakes are applied. These factors in turn lead to the necessity of determining a conservative perception and brake reaction time and the establishment of a proper coefficient of friction between tires and road surface. Although not generally conceded, the latter factor is influenced by climatic conditions, for where ice and snow are prevelent such conditions should be given consideration up to the maximum speed that a motor car can operate on ice on tangent. These factors are illustrated in Figure 8 while Figure 9 shows the traction requirements for a vehicle operating on a tangent on level grade at various speeds. It will be noted that at a speed of 60 M.P.H. a traction of approximately 0.08 is required which is nearly the maximum friction available on ice and this illustrates that 60 M.P.H. is about the maximum speed possible on a straightaway on icy surfaces.

In the determination of daylight rear end collision sight distance, the engineers of the Pennsylvania Turnpike Commission adopted a perception-reaction time of 2 seconds and a coefficient of friction of f=0.40; and established a sight distance based on a height-of-eye-of-driver, amounting to 4.5 feet, to an object 4 inches high at a speed of 70 M.P.H. This factor resulted in a rear end collision sight distance of 600 feet at 70 MP.H. The sight based on an object 4 inches high is of interest and

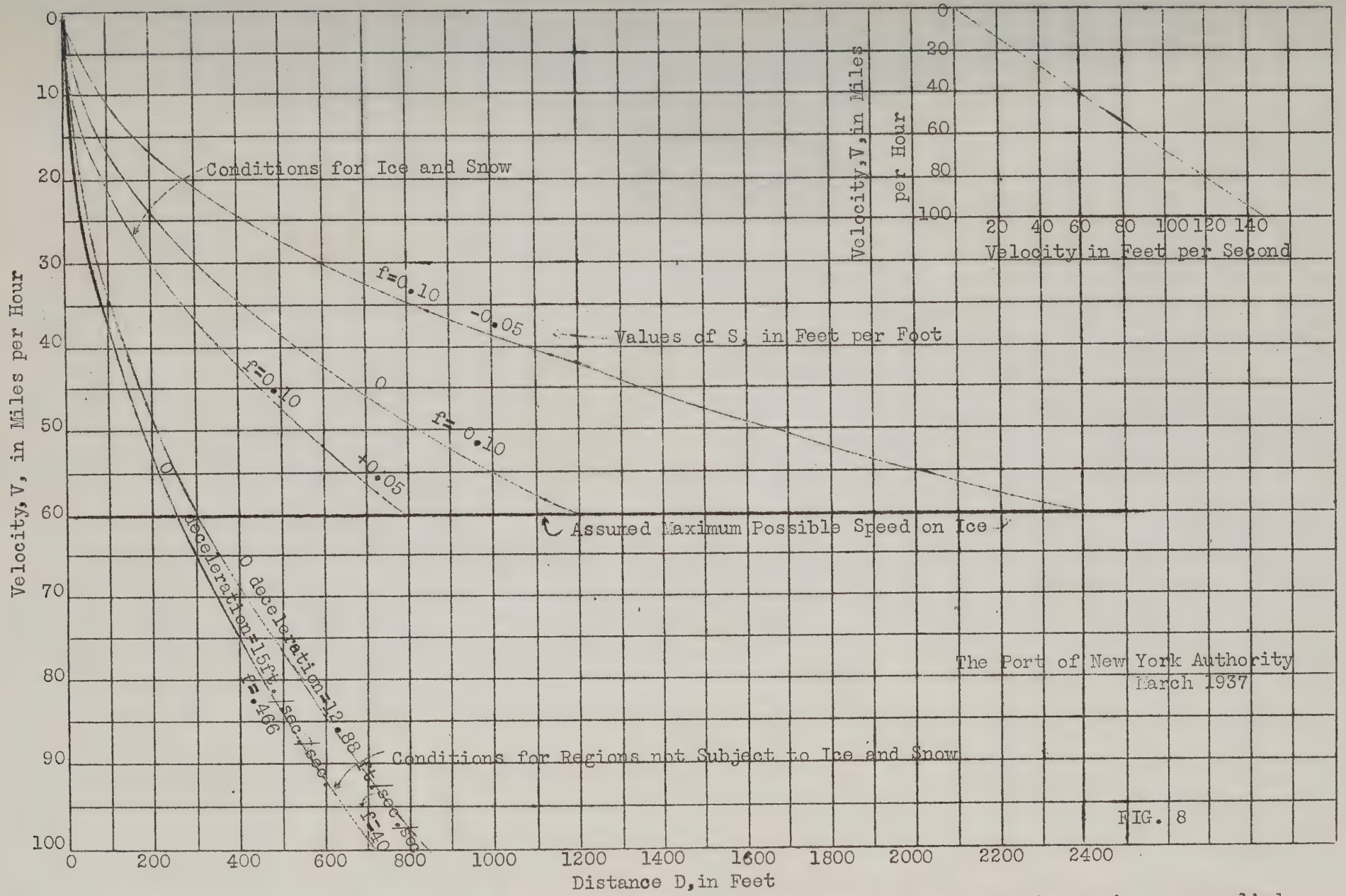


FIG. 8

Distance Required to Bring a Motor Car Equipped with Four Wheel Brakes to a Stop After Brakes are applied.

FRICTION COEFFICIENTS FOR TRACTION - BASED ON ROLLING & AIR RESISTANCE (ENGINE RESISTANCE NOT INCLUDED)

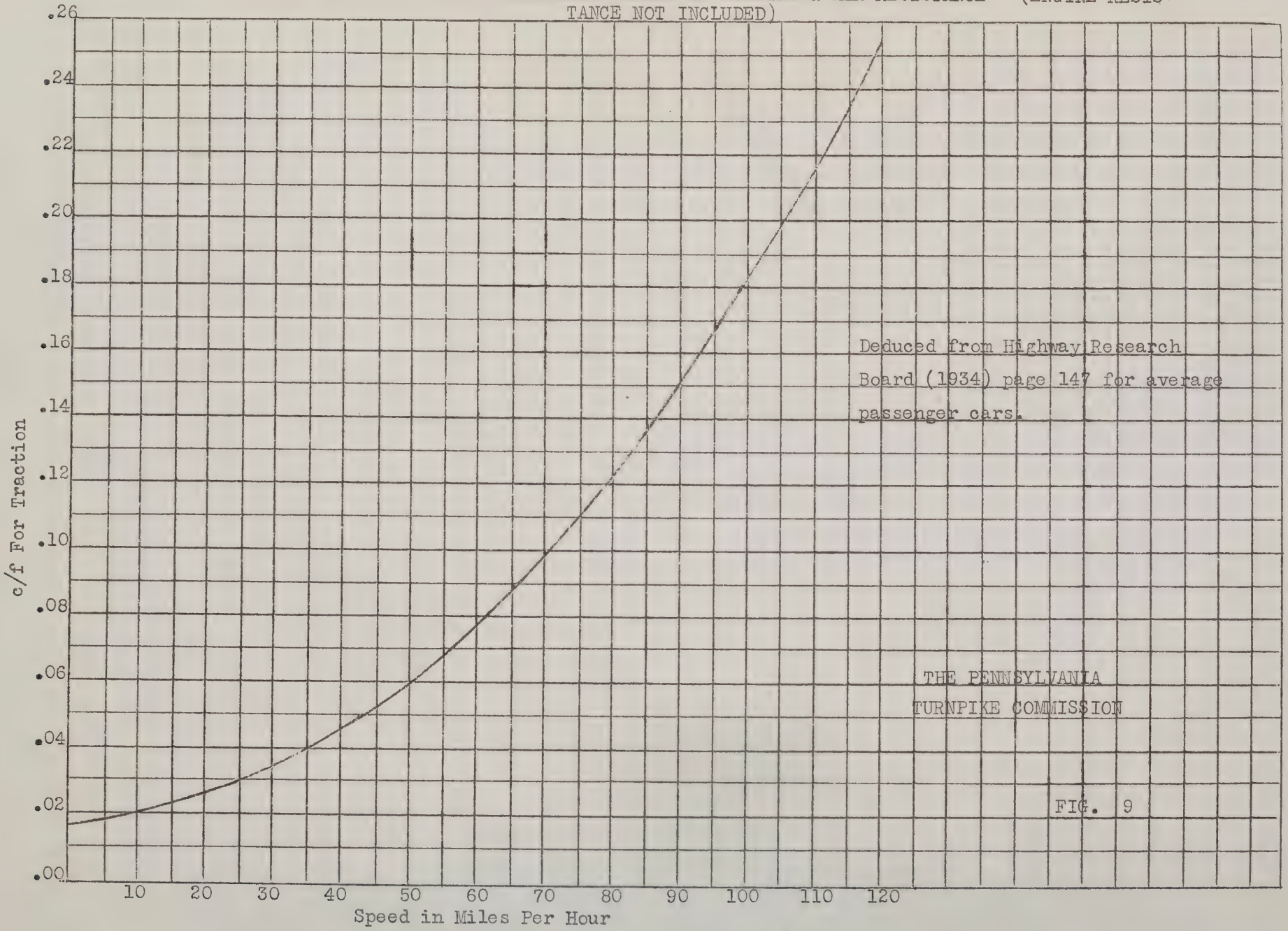


FIG. 9

immediately introduces the question as to whether this criterion rather than car to car sight should be applied to higher design speeds.

There are several practical limitations involved in doing this, however, the limit of vision of the human eye, and economic considerations. This latter is apparent when the length of vertical curves and the amount of horizontal curve widening is investigated.

Although there is no substantiating data it is the writer's belief that two factors should be utilized in the determination of daylight rear end collision sight distance, the line of vision to a 4 inch object within the limitations of visibility to the human eye, and car to car vision (4.5 feet to 6 feet) for the full design speed; taking icy factors into consideration where warranted by climatic conditions. It is quite possible that 600 feet is the maximum distance that a 4 inch object can be seen clearly by the human eye, but to the writer's knowledge no tests have been made to definitely establish this fact.

It will be noted that a perception and reaction time of two seconds and an allowable braking friction factor of f=0.40 for non-icy surfaces appears to meet the approval of highway engineers generally, but the extent to which ice should be taken into account and whether the vision should be from car to car, or from car to a 4 inch object, is quite unsettled. With a rear end collision condition, the possibility of the car being able to turn aside from the obstruction before becoming involved in a collision furnishes a factor of safety not always present in the case of passing sight distance on two lane two way highways. However, it is pertinent to note that car to car sight distance (neglecting ice) can be provided almost as economically for a speed of 100 MPH as sight distance to a four (4) inch object (neglecting ice) for a speed of 70 MPH, except where horizontal and vertical curvature are combined. Table I shows the braking plus reaction distance required for various speeds.

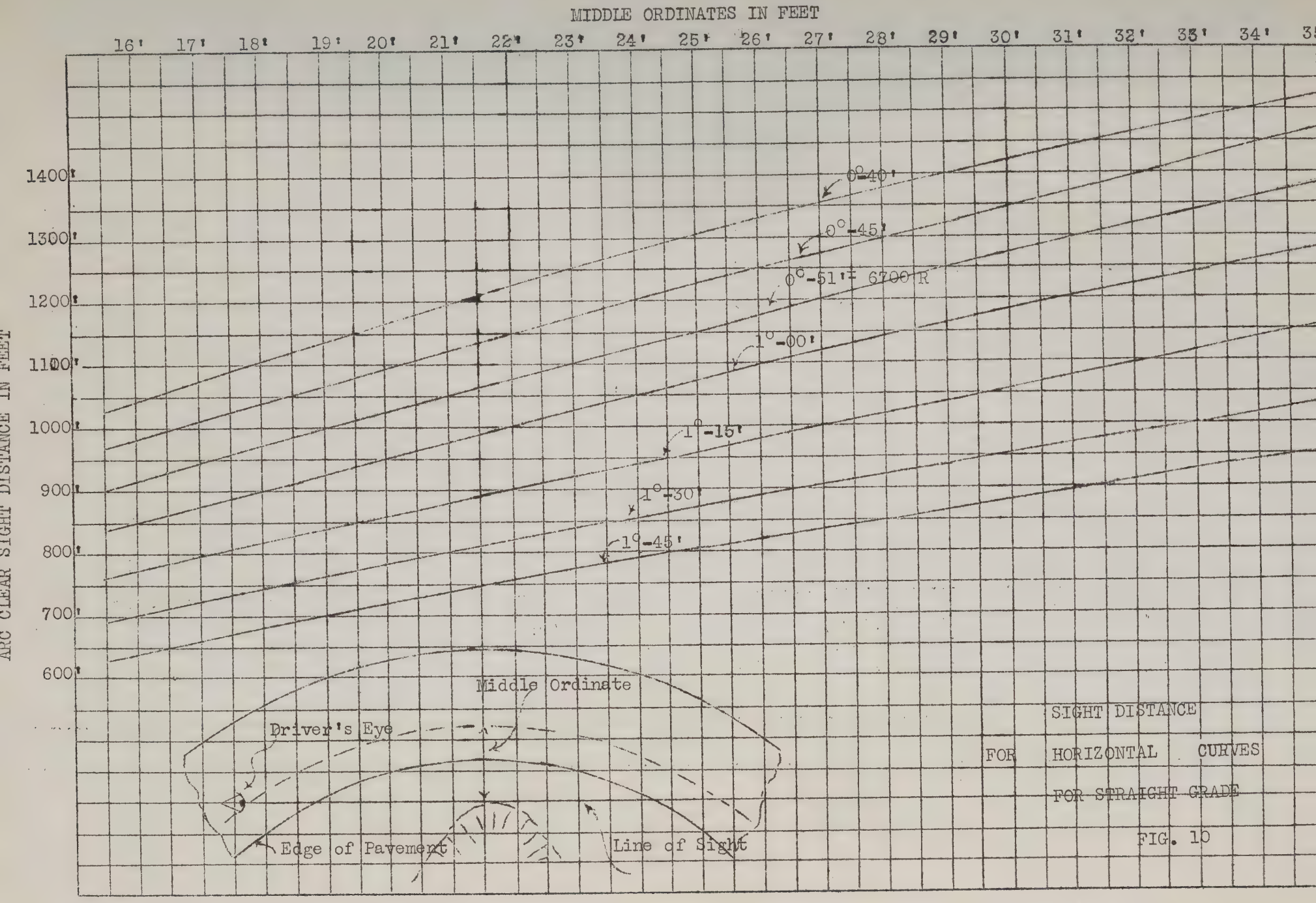
After the establishment of sight distance criteria it is necessary to apply the data to vertical curvature, horizontal curvature and to both in combination where such combination occur, taking into consideration the presence of cut slopes, shrubbery,

TABLE I

REAR END COLLISION SIGHT DISTANCE BASED ON BRAKING ON A LEVEL GRADE

	Dry and Wet Pavements								Icy Pavement					
	Daylight Condition				Night Condition				Daylight Condition			NIGHT CONDITION		
Speed M.P.H.	Perception and Reaction Time Secs.	Feet	Braking Distance (f=0.40) in Feet	Req'd Sight Distance Feet	Perception and Reaction Time Secs.	Feet	Braking Distance (f=0.40) in Feet	Req'd Sight Distance Feet	Perception and Reaction Time Seconds	Braking Distance (f=0.10) in Feet	Req'd Distance Feet	Perception and Reaction Time Seconds	Braking Distance (f=0.10) in Feet	Req'd Sight Distance Feet
100	2	292	840	1132	3	438	840	1278						
90	2	264	680	944	3	396	680	1076						
80	2	234	540	774	3	351	540	891						
75	2	220	470	690	3	330	470	800						
70	2	206	410	616	3	309	410	719						
65	2	191	350	541	3	286	350	636						
60	2	176	300	476	3	264	300	564	2	1200	1376	3	1200	1464
55	2	162	255	417	3	243	255	498	2	1000	1162	3	1000	1243
50	2	146	208	354	3	219	208	427	2	830	976	3	830	1049

Note: No data for speeds greater than 60 M.P.H. for icy conditions is shown because it is assumed that it will be practically impossible for a motor car to operate at a speed greater than 60 M.P.H. on ice.



SIGHT DISTANCE

FOR HORIZONTAL CURVES

FOR STRAIGHT GRADE

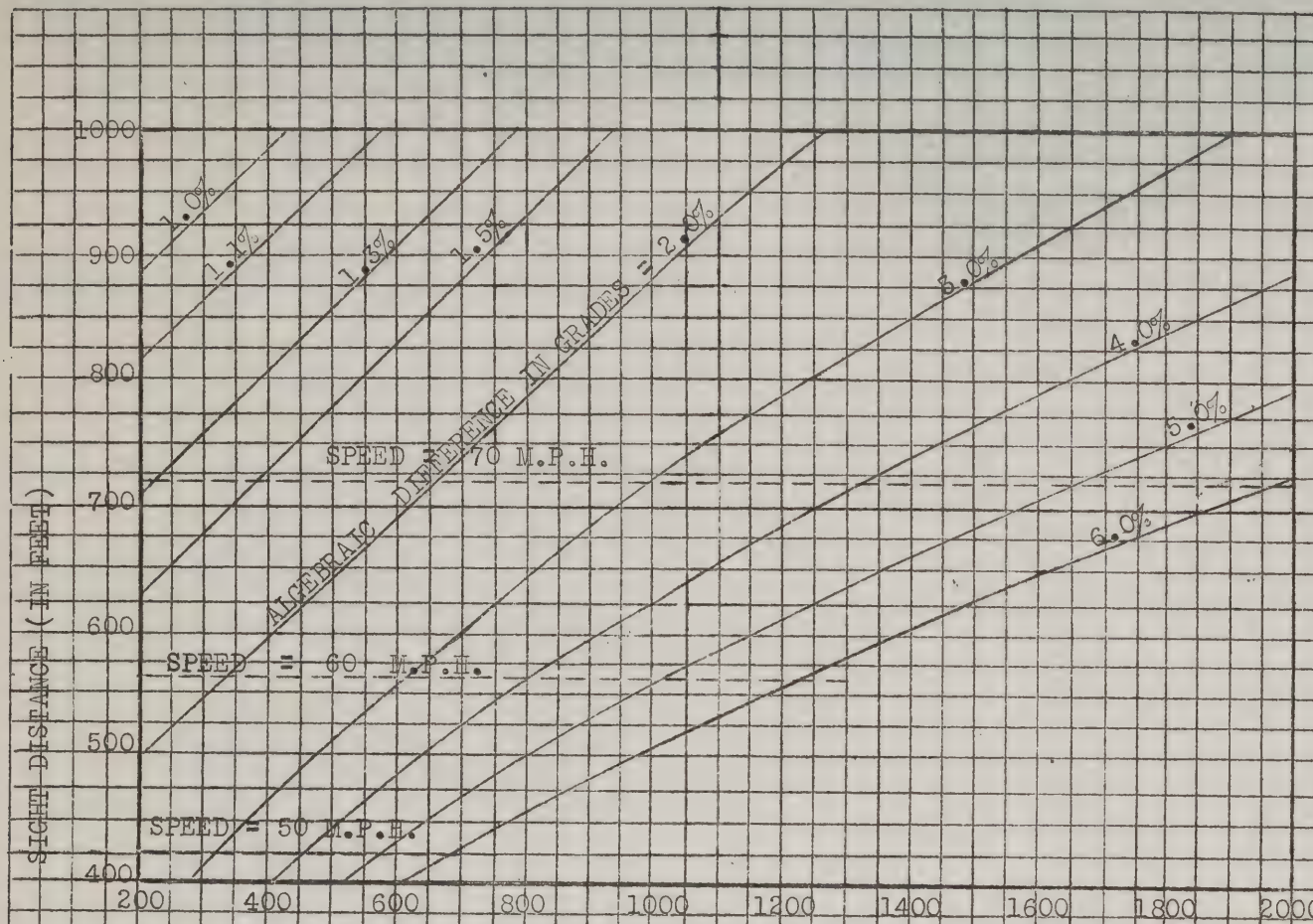
FIG. 10

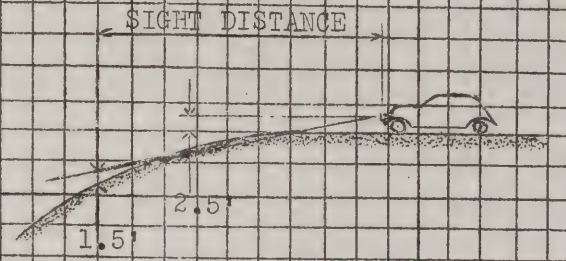
and trees. Figure 10 shows the effect of horizontal curvature on sight distance.

Rear end collision sight distance at night presents problems in addition to those involved under daylight conditions inasmuch as the characteristics and limitations or motor car headlamps as well as slower human reactions enter the picture.* At night, it is desirable to illuminate the pavement surface to the limit of the capacity of present headlamp equipment and to make an additional allowance for future improvements in such units. Furthermore, because it has been demonstrated by several thousand tests that human perception is slower at night, it is necessary to increase the allowance for perception and brake reaction time in determining rear end-collision sight forheadlamp visibility during the hours of darkness.

After considerable study of the meager data available a perception-reaction interval of 3 seconds was adopted for the Turnpike; and it was considered reasonably satisfactory, in view of the material expense involved, if the pavement were not a greater distance than 1½ feet below the light rays at the extreme limit of the sight distance when summit vertical curves were involved. It must be recognized that although objects as high as a man or dog may be seen as far away as 1,000 feet with properly adjusted headlamps, nevertheless the surface of the pavement is shadowy and indistinct at that distance, therefore, in view of the absence of supporting tests, the provision of summit vertical curves of sufficient length to place the surface of the pavement in the light plane for the entire sight distance appears unwarranted, at least until more facts are made available. The Pennsylvania Turnpike offers a unique opportunity for the performance of tests with the new sealed beam headlamps so that basic facts may be recorded. Figures 11 and 12 show the lengths of vertical curves required for headlamp distance on summits and sags respectively.

* "The Relationship of Headlamps to Road Speed", by W. C. Giessler, Illinois Division of Highways, (February 1937)
"What is the Night Accident Problem Like", by W. C. Giessler (May 1937)
"Seeing with Motor Car Headlamps" Transactions of the Illumination Engineering Society (May 1938)





DESIRABLE SIGHT DISTANCE=1000 FT.

MINIMUM SIGHT DISTANCE REQUIRED
50 M.P.H. 425 FT.
60 M.P.H. 565 FT.
70 M.P.H. 720 FT.

MINIMUM LENGTH OF VERTICAL CURVES ON SUMMITS (IN FEET)

SUMMARY OF INSTRUCTIONS

(a) Provide 1000-foot headlamp sight in easy country on tangent sections at sags, and minimum sight for a speed of 70 MPH at summits.

(b) Check speed on horizontal curves for speed based on f=0.10 and limit headlight sight to such speed.

(c) Check effect of cut slopes and adjust headlamp sight accordingly.

(d) Check effect of horizontal spread of light on horizontal curves and adjust headlamp sight accordingly.

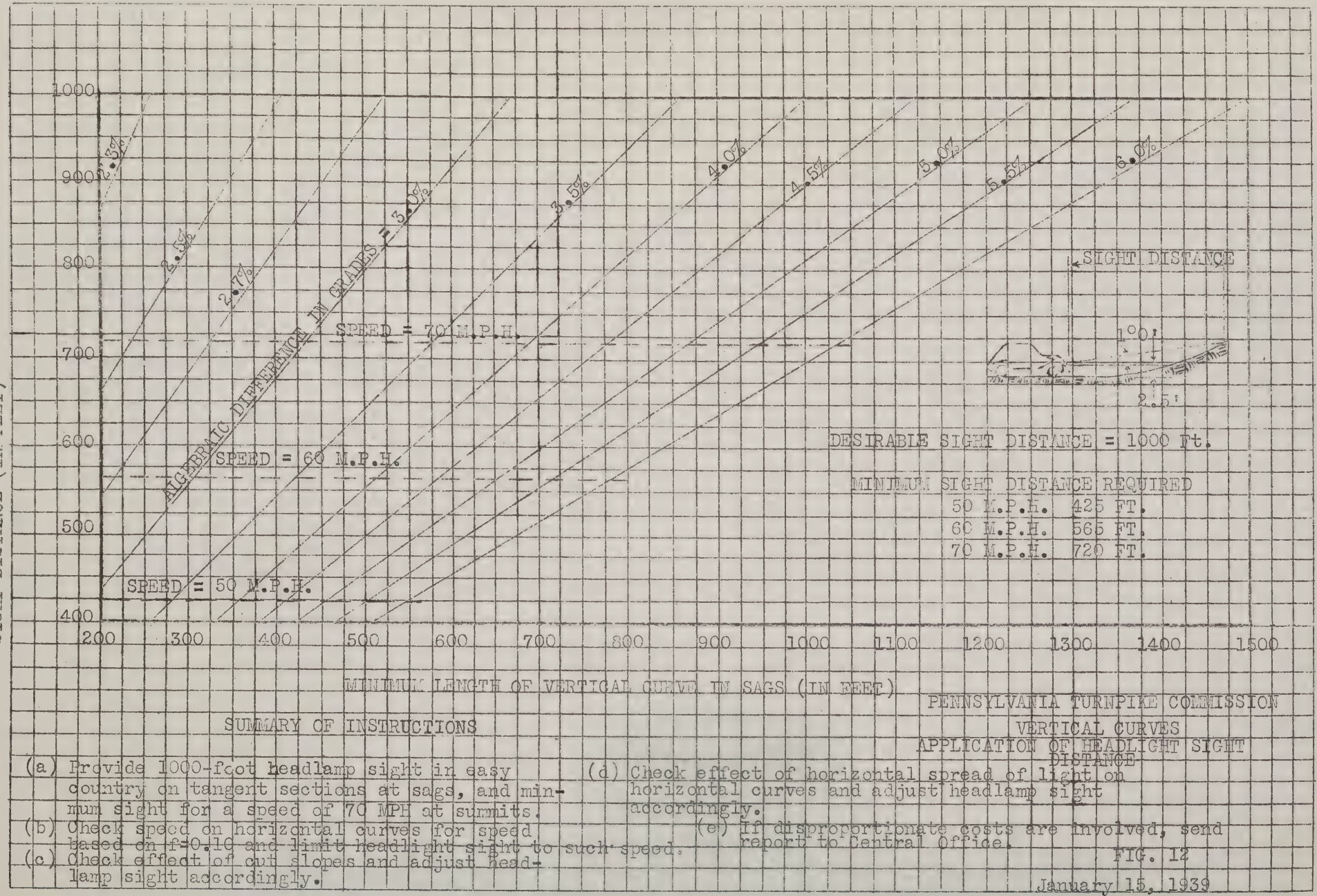
(e) If disproportionate costs are involved, send report to Central Office.

PENNSYLVANIA TURNPIKE COMMISSION

Vertical Curves FIG. 11

Application of Headlight Sight Distance

January 15, 1939



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(e) If disproportionate costs are involved, send report to Central Office.

PENNSYLVANIA TURNPIKE COMMISSION

VERTICAL CURVES

APPLICATION OF HEADLIGHT SIGHT DISTANCE

FIG. 12

January 15, 1939

Because the lateral spread of headlamps is a limiting factor to night time seeing on horizontal curves an investigation was made by the Turnpike engineers to determine just what effect this limiting factor would have on sight distance. No information was available but correspondence with Mr. Val. J. Roper of General Electric Co. resulted in obtaining sufficient information to enable the preparation of Figure 13. The same intensity at the fringes of the lateral spread was used which is available at a distance of 1,000 feet directly ahead of headlamps. It will be noted that horizontal curvature limits visibility with present automotive headlamp equipment but the possibility of improvement in lamp design which will overcome the weakness should be recognized. The data shown in Figure 13 w as based on 1940 units, but the information was made available in March 1939, and there is a possibility that the lamps now in actual use do not conform precisely with the data furnished.

Sight distance requirements have an important bearing on the total cost of a project in addition to the effect on safety of vehicle operation and it is hoped that a uniform and consistent policy may be developed in the near future.

Horizontal Curvature and Superelevation

As previously stated, the speed potentialities of the modern motor car makes it necessary to enlarge design thinking in order that the highway may not be obsolete before it is completed. This is particularly true in the case of horizontal curvature. Statements have been made in various parts of the country that certain highways are being designed for a speed of 100 MPH, but upon examination it is found that such a speed is hypothetical because the assumption is made that a motor car can successfully develop an unbalanced side friction factor of f=0.30. The truth of the matter is, that no one knows what friction factor can be safely developed on curves at that speed. Barnett of the Public Roads Administration hazards the guess that it may be 0.08*.

There are two schools of thought which has divided engineering opinion on the question of design speed and horizontal curvature. One school favors setting up the

* Highway Research Board (1936) p.69

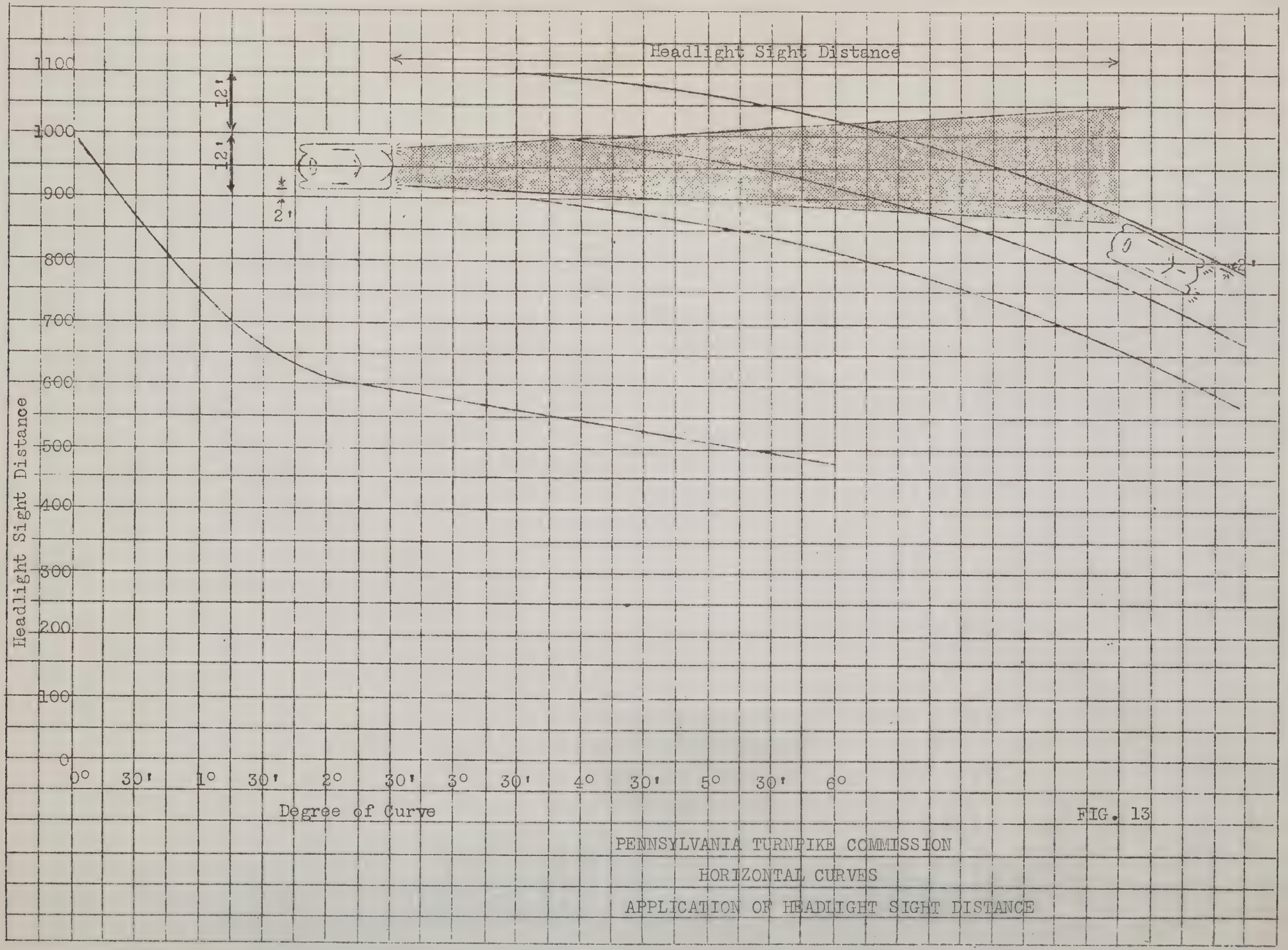


FIG. 13

PENNSYLVANIA TURNPIKE COMMISSION

HORIZONTAL CURVES

APPLICATION OF HEADLIGHT SIGHT DISTANCE

full deign speed, as for example 100 MPH, based on conservative friction factors and limiting curvature in relation thereto; the other proposes using a more moderate speed for curve limitation, for example 70 MPH, based on conservative friction factors and then examining the friction factors to make sure they do not exceed a certain amount at a higher speed known as the critical speed.

At the moment, engineers are at a disadvantage when attempting to establish sound and authoritative criteria for horizontal curvature because very little is known of permissible friction factors and vehicle operation on curves at speeds above 70 MPH. As a matter of fact, where high speed is concerned the radii of curves becomes so large that the problem, long known to railway engineers, arises of superelevation to care for the conflicting requirements of slow moving and fast traffic. In railway practice the problem is one of rail wear and has been met by adjusting the superelevation so that fast trains will wear the outer rail the same amount that slow trains wear the inner. In highway design the problem is not one of wear, but of safety, for while a railway is able to enforce a slow order on a curve, such a procedure has not proven practicable with the American motoring public; and if a flat curve is superelevated sufficiently for high speed operation, slower moving vehicles, such as large trucking units, may be induced unconsciously to "edge in", because of the forces of gravity, and crowd the inner lane, possibly with disasterous results to other vehicles. This suggests that the radius of the sharpest permissible curve should be sufficiently expanded so that it is possible to utilize a very moderate maximum rate of superelevation.

A series of studies depicted on speed diagrams will illustrate the problems involved in high speed curve design and superelevation.

Figure 14 was prepared based on the assumptions that the unbalanced friction due to centrifugal force should not exceed f=0.07, and a maximum rate of superelevation of 0.03 feet per foot of width. The maximum superelevation was limited to 0.03 feet in an effort to satisfy the conflicting objectives inherent in operation at high and low speeds. Four loci are plotted on the diagram to show the speed-

radius-superelevation

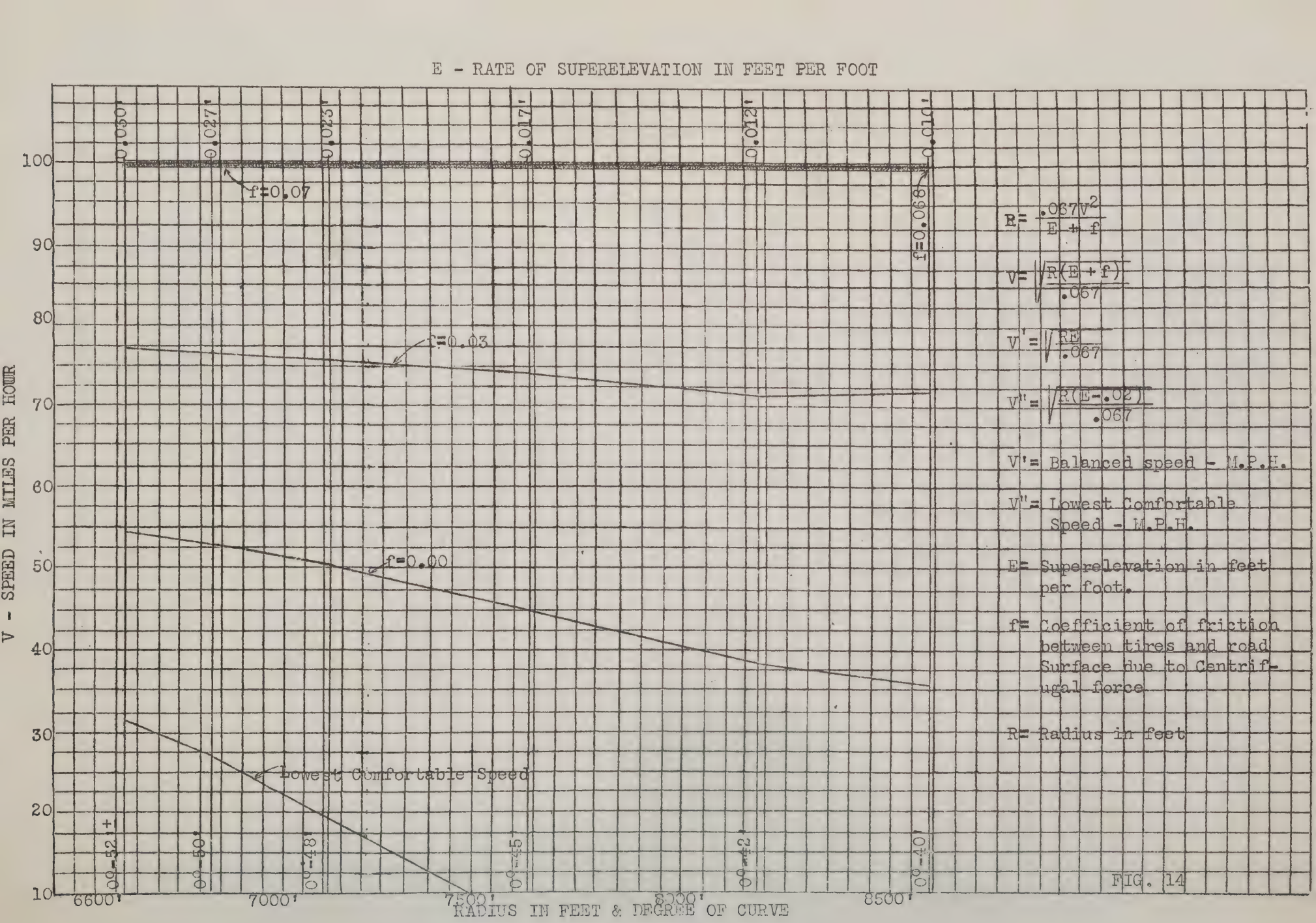

E - RATE OF SUPERELEVATION IN FEET PER FOOT
0.030'
0.027'
0.023'
0.017'
0.012'
0.010'
f=0.07
f=0.068
f=0.03
f=0.00
Lowest Comfortable Speed
V - SPEED IN MILES PER HOUR
100
90
80
70
60
50
40
30
20
10
0°-52'+
0°-50'
0°-48'
0°-45'
0°-42'
0°-40'
6600'
7000'
7500'
8000'
8500'
RADIUS IN FEET & DEGREE OF CURVE
R= .067V²/(E + f)
V= √(R(E + f)/.067)
V'= √(RE/.067)
V''= √(R(E-.02)/.067)
V'= Balanced speed - M.P.H.
V''= Lowest Comfortable Speed - M.P.H.
E= Superelevation in feet per foot.
f= Coefficient of friction between tires and road Surface due to Centrifugal force
R= Radius in feet

FIG. 14

relationship for f=0.07, f=0.03 (assumed icy surfaces), f=0.0 (balanced forces) and f=0.02 (assumed lowest comfortable speed). At 100 MPH the speed locus is horiztonal at a constant friction value of f=0.07 until the radius has expanded to 8500 feet. For radii larger than 8500 feet the friction willgradually fall because a super-elevation less than 0.01 feet would be required to hold the friction constant, whereas practical drainage requirements prohibit a cross slope of less than 0.01 feet. The locus for icy surfaces is of a good shape indicating uniform and consisting operating conditions, but the speed range is considerably above the maximum speed at which it is assumed a vehicle can operate on ice. The locus for balanced forces indicates uniform and consistent operating conditions, but the sharpest curve requires a speed of 55 MPH which is too high for heavy trucking units where long grades over 2% are involved. It will be noted that with the above stated assumptions, curvature cannot be sharper than 6670 feet radius (0° 52'). In other words, if the assumptions used are correct, it will not be possible to really obtain 100 MPH highways where the radius of curvature must be sharper than 6670 feet.

Exploring the ideas of the other school of thought, which promotes the policy of designing for more moderate speeds but assumes also that safety is provided at a higher or critical speed, a diagram, Figure 15, is presented. The design speed is assumed at 70 MPH and the maximum superelevation at 0.03 feet. This expedient results in permitting curvature as sharp as $1^{\circ}45'$, and has considerably reduced the speed required to obtain a balancing of forces, which is of a distinct advantage to slow moving vehicles. The loci for f=0.03 and f=0.0 are not as well shaped as in Figure 14 and consequently do not offer, theoretically, as consistent operating conditions as might be desired. The most serious drawback is the amount of friction developed at 100 MPH)f=0.172), for, to the writer's knowledge there is no evidence or operating experience to prove that this friction is available at such a high speed, much less that it offers safe operating conditions. It would therefore appear that engineers are on unsafe ground, when designing a 100 MPH highway in assuming that is it permissible to utilize a friction factor as high as f=0.17 until tests

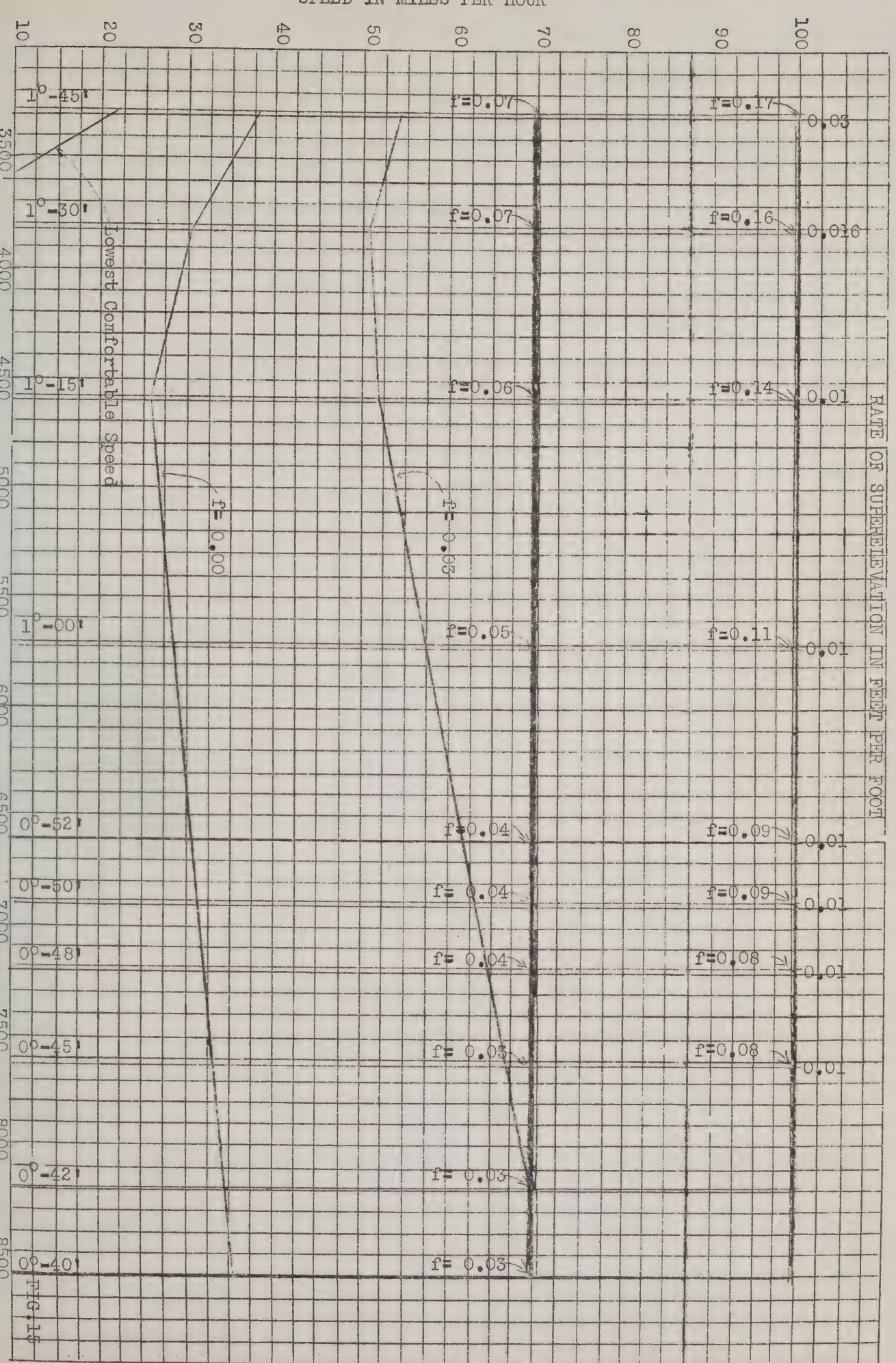

SPEED IN MILES PER HOUR
10
20
30
40
50
60
70
80
90
100
RADIUS IN FEET & DEGREE OF CURVE
3500
4000
4500
5000
5500
6000
6500
7000
7500
8000
8500
1°-45'
1°-30'
1°-15'
1°-00'
0°-52'
0°-50'
0°-48'
0°-45'
0°-42'
0°-40'
Lowest Comfortable Speed
f=0.07
f=0.07
f=0.06
f=0.05
f=0.04
f=0.04
f=0.04
f=0.03
f=0.03
f=0.00
f=0.03
f=0.17
f=0.16
f=0.14
f=0.11
f=0.09
f=0.09
f=0.08
f=0.08
0.03
0.016
0.01
0.01
0.01
0.01
0.01
0.01
RATE OF SUPERELEVATION IN FEET PER FOOT

FIG. 15

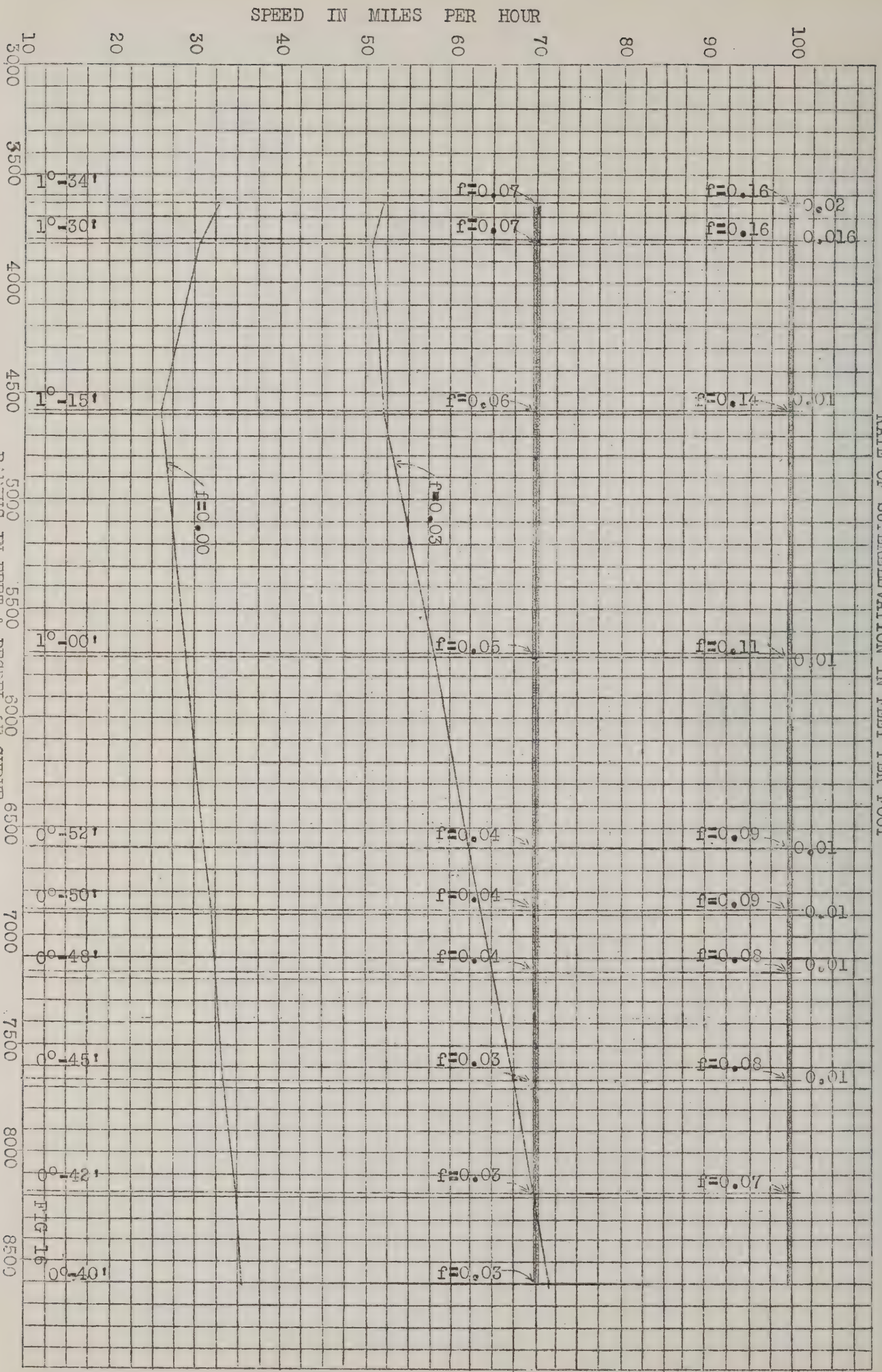

SPEED IN MILES PER HOUR
10
20
30
40
50
60
70
80
90
100
RADIUS IN FEET & DEGREE OF CURVE
3000
3500
4000
4500
5000
5500
6000
6500
7000
7500
8000
8500
1°-34'
1°-30'
1°-15'
1°-00'
0°-52'
0°-50'
0°-48'
0°-45'
0°-42'
0°-40'
f=0.07
f=0.07
f=0.06
f=0.05
f=0.04
f=0.04
f=0.04
f=0.03
f=0.03
f=0.03
f=0.16
f=0.16
f=0.14
f=0.11
f=0.09
f=0.09
f=0.08
f=0.08
f=0.07
0.02
0.016
0.01
0.01
0.01
0.01
0.01
0.01
f=0.00
f=0.03
RATE OF SUPERELEVATION IN FEET PER FOOT

FIG 16

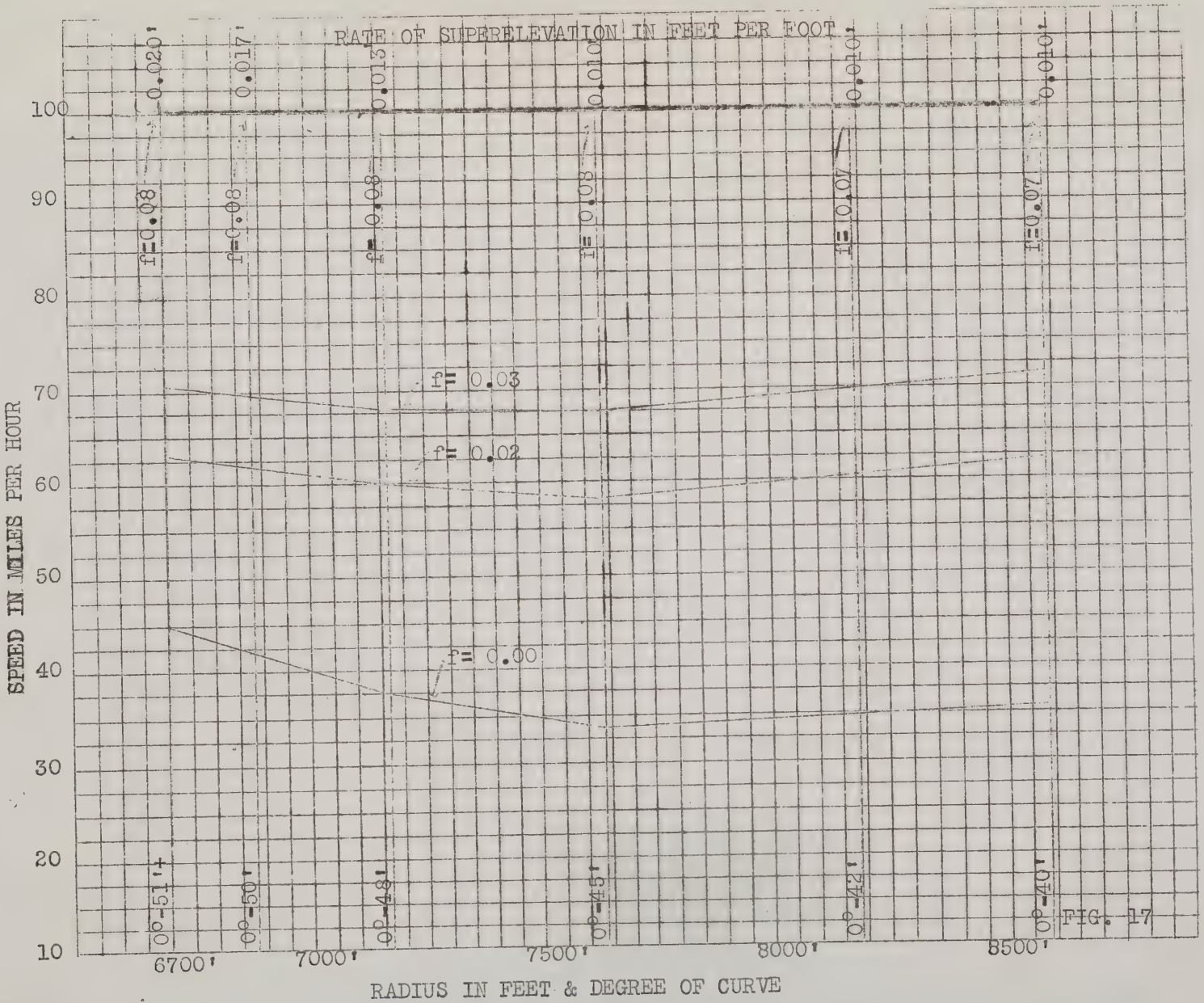

RATE OF SUPERELEVATION IN FEET PER FOOT
0.020'
0.017'
0.013'
0.010'
0.010'
0.010'
f=0.08
f=0.08
f=0.08
f=0.08
f=0.07
f=0.07
f= 0.03
f= 0.02
f= 0.00
SPEED IN MILES PER HOUR
100
90
80
70
60
50
40
30
20
10
0°-51'+
0°-50'
0°-48'
0°-45'
0°-42'
0°-40'
6700'
7000'
7500'
8000'
8500'
RADIUS IN FEET & DEGREE OF CURVE

FIG. 17

with actual stock models furnish proof. Figure 16 is similar to Figure 14 except that the maximum rate of superelevation was set at 0.02 feet and was prepared in order to study the speed conditions for f=0.0.

Returning to the principle of designing for the maximum speed, but modifying the assumptions somewhat in an effort to improve the conditions for f=0.0, Figure 17 resulted. The maximum s uperelevation was limited to 0.02 feet (approximately $\frac{1}{4}$" per foot) and the maximum friction at 100 MPH to f=0.08. The loci appear reasonably well shaped and present consistent operating conditions from a practical standpoint. The speed conditions at f=0.0 are considerably more satifactory than in Figure 14. It is interesting to note the minimum radius has only increased from 6670 feet (Figure 14) to 6700 feet.

A thoughtful study of Figures 14 to 17 inclusive leads to the realization that more knowledge is required before curve design can be approached with confidence. In view of the indication that 100 MPH highways are an impossibility unless curves of 6700 foot radius, or flatter, are used it would appear imperative that a test program be initated in order that authoritative data be made available so that design can proceed with confidence and that there may be no economic waste. It would be unfortunate if curve design were based on erroneous assumptions and if large sums were expended to bring other features of the design, such as sight distance, up to the standards of a hypothetical speed; and if later it were discovered that the safe and practicable operating speed on the curves was much lower than assumed. Such unbalance in design would necessarily result in an economic loss.

As a matter of general interest, it may be noted that where sharp curvature and high rates of superelevation are used there appears to be a tendency on the part of drivers to run up higher friction values on curves which are highly superelevated. It has occured to the writer that there may be some physical reason why drivers will naturally run up a higher friction factor on such highly superelevated curves and the suggestion is offered that it may be due to the difference in center of gravity of the body of the driver and that of the vehicle and that, therefore, the extra

tilt

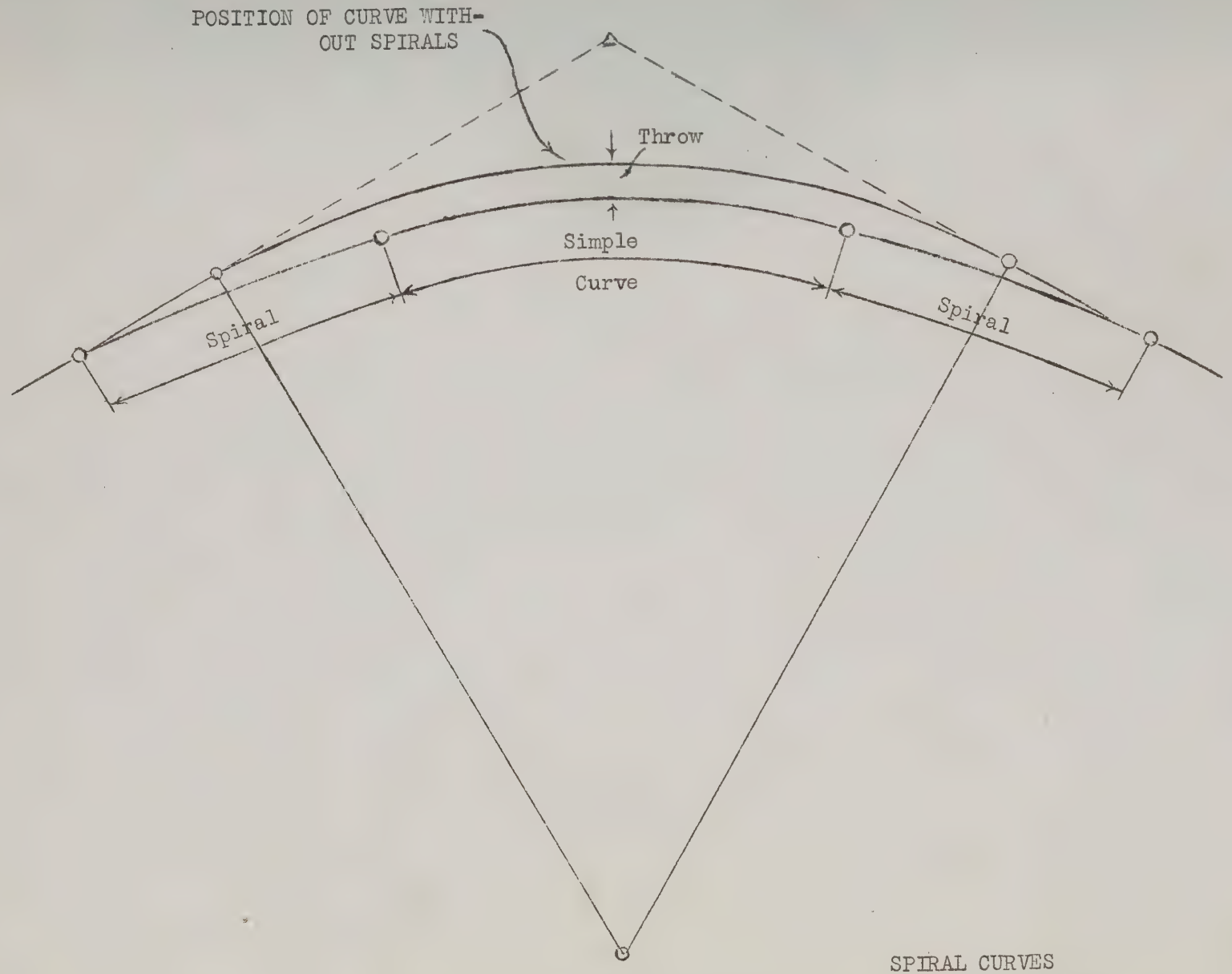
in superelevation enables the driver to increase the friction factor without a corresponding increase in a sense of danger. This is, of course, somewhat contrary to present accepted ideas on the subject. In considering this phenomena it must be remembered that the position of the driver in the seat and in relation to the steering wheel has an important bearing on the ability to negotiate a curve when developing a high friction factor.

Spirals

The use of spiral curves in highway practice is, with a few exceptions, relatively new. The spiral in universal use was developed for railways and is based on the logical reasoning that the rate of spiral curvature should increase uniformly from zero to the degree of the simple curve, thus enabling the superelevation to increase in a straight line from zero to a maximum rate, so that the superelevation will exactly satisfy the requirements of the radius at every corresponding point. It also permits a uniform rate of change in centrifugal acceleration, provided the balancing affect of superelevation is neglected. The curve which satisfied those conditions is a cubic equation and all present spirals are of this form regardless of the methods of approximation which have been developed for field stakeout. Figure 18 illustrates the general principles involved in relation to the simple curve.

Two schools of thought have developed in the method of determining the length of spiral to use, the rate of rotational change per unit of time to accomplish the maximum superelevation, and the rate of change of centrifugal acceleration (neglecting the balancing forces due to superelevation). The former method is generally used by the railways and their experience indicates a conservative rate of 2% of cross slope per second of time at the assumed speed, while the latter was proposed by Royal-Dawson* and has been accepted by Moyer**, the Public Roads Administration and various State Highway Departments. Moyer recommends a rate of change of centrifugal acceleration of 2 with a resulting formula of $L = \frac{1.58V3}{R}$, in which V equals the design speed in miles per hour and R equals the radius of the simple curve in feet.

* Royal-Dawson, F. G. - Elements of Curve Design, etc. (London 1932)
** Bulletin 120, Iowa Experiment Station, Ames, Iowa



SPIRAL CURVES

FIG. 18

So far as is known, no one has questioned the validity of a cubic equation for highway use and this is due, in all probability, to the fact that it is theoretically correct. Actual operating experience has shown that it is correct from a practical standpoint for railway conditions, where the vehicle must accurately follow the spiralled path because of the rails; and it may be correct for highway usage for curves sharper than 4°.

In considering any feature of highway design it must be constantly remembered that highway operation is composed of two parts, the vehicle, and the operator. The vehicle being "free wheeled" and the operator a free agent imposes the task of providing a curve with a locus which will fit the practical limitations of both and provide a path conforming to the natural tendencies of vehicle and operator. When considering spirals for flat curves, the cubic equation provides a locus with a radii of almost infinity along the first 1/3 of its length, and it would appear practically an impossibility for an operator to turn the wheel such an infinitely small amount. Casual observation also indicates that most operators utilize more "throw" than is porvided by a cubic equation. An example will illustrate the two methods of determining spiral length and indicate the order of magnitude of the component parts of a spiral for a flat curve under the following assumptions:

Design speed	100 MPH
Radius of curve	6670 feet
Rate of superelevation	.03 feet per foot of width
Pavement slope on tangent	.01 feet per foot of width

Utilizing the method of rotation at the rate of 2% cross slope per second of time, a spiral 294 feet long will result, which for practical reasons would be rounded up to 300 feet. The "throw" for a 300 foot spiral amounts to 0.56 feet.

Utilizing the method of the rate of change of centrifugal acceleration with a rate of 2 feet per second per second per second, a spiral 236 feet long will result, which for practical reasons would be rounded up to 240 feet. The "throw" for a 240 foot spiral

amounts to 0.36 feet.

Considerable imagination is required to lead one to hope that the car can be so delicately manipulated by the driver that a path can be so accurately followed in a distance of 300 feet that a "throw" of only 6-3/4 inches is attained, and yet if the path is not accurately followed the theory of the cubic equation breaks down. It is doubtful if a vehicle can be operated this accurately on a tangent in moderately windy weather, while in a gusty wind a car traveling at 70 MPH and upwards is moved laterally several feet in distance less than 300 feet. It is therefore felt that more "throw" is required than indicated by a cubic equation.

Because of the unknown and controversial issues involved in the design of spiral curves the most logical procedure would be to conduct a series of accurately controlled and instrumented tests in order to establish natural travel paths selected by operators in entering curves and to discover what turning radius results from the slightest pressure on the steering wheel so that this detail of design can be established on a firm practical basis. The Pennsylvania Turnpike offers an unequaled proving ground for the performance of such tests and the hope is expressed that advantage will be taken of the opportunity before the Turnpike is opened to the public, after which time a test program will not be feasible.

LENGTH OF TANGENTS

Design speed has an effect on the length of tangent between horizontal and vertical curvature but as yet no data is available from which satisfactory criteria may be set up. There can be little question but that the time element is the governing feature but no one can state authoritatively whether 5, 10, 15 or 20 seconds is the correct value to apply. If the time factor becomes known the problem would at once resolve itself into simple arithmetic. For instance if 10 seconds is assumed as the minimum desirable period between horizontal or vertical curvature a minimum of 1500 feet would result for a design speed of 100 MPH. For profiles where vertical curves 2000 feet in length are used the distance between points of vertical intersection would amount to 3500 feet. This illustrates the economic importance of the problem for projects

located in rolling and hilly territory.

ENTRANCE AND EXIT POINTS

It is undesirable to space traffic interchange facilities too close together in terms of time because of the turbulence occasioned by such traffic movements and of the confusion to the motoring public due to "flashing" by exits every few minutes. If the highway were designed for 50 MPH, a spacing of 10 miles would mean passing an exit every 12 minutes while at 100 MPH the exits would "flash" by every 6 minutes.

The length of deceleration and acceleration lanes are a function of design speed for it is desirable that a deceleration lane be of sufficient length that a vehicle may slow down sufficiently by engine compression alone to make a turnout safely, while the acceleration lane should be of sufficient length to permit an entering vehicle to accelerate to a speed somewhat comparable to the design speed, as well as to place it in view of the main traffic several seconds before encroachment within the high speed lanes, in order to eliminate the element of surprise.

The shape of the turnout will be influenced by the design speed because of the tendency of the motorist to enter a turnout at too high a rate. This tendency is caused by the inability of the average motorist to accurately judge the speed of a slowdown after having operated at high speed for a period of time. Fig. 19 illustrates the method of solving turnout design for Turnpike conditions.

SIGNS

The design, size or lettering, number of words in message and position of a direction sign depend on design speed. With a known design speed the factor of time can be entered in the equation and a solution obtained provided certain knowledge respecting reading and reaction time is made available by a test program. Frequently a motorist passes a direction sign before being able to read it, or if the letters of the sign are of sufficient size to be read, it may be placed improperly in such a position that the motorist has passed a turnout before he has had time to react to make the turn. For example, if the speed is assumed as 60 MPH and the reading and

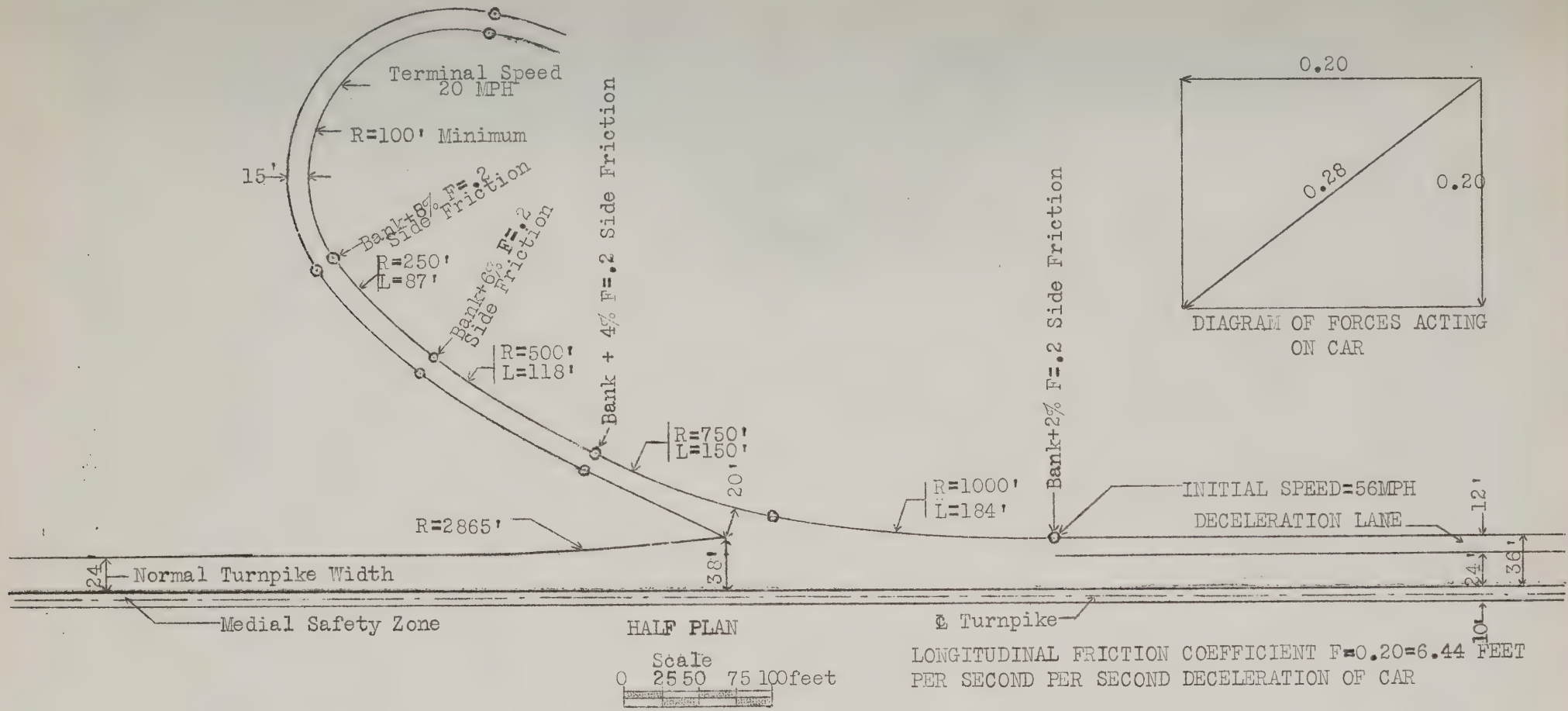
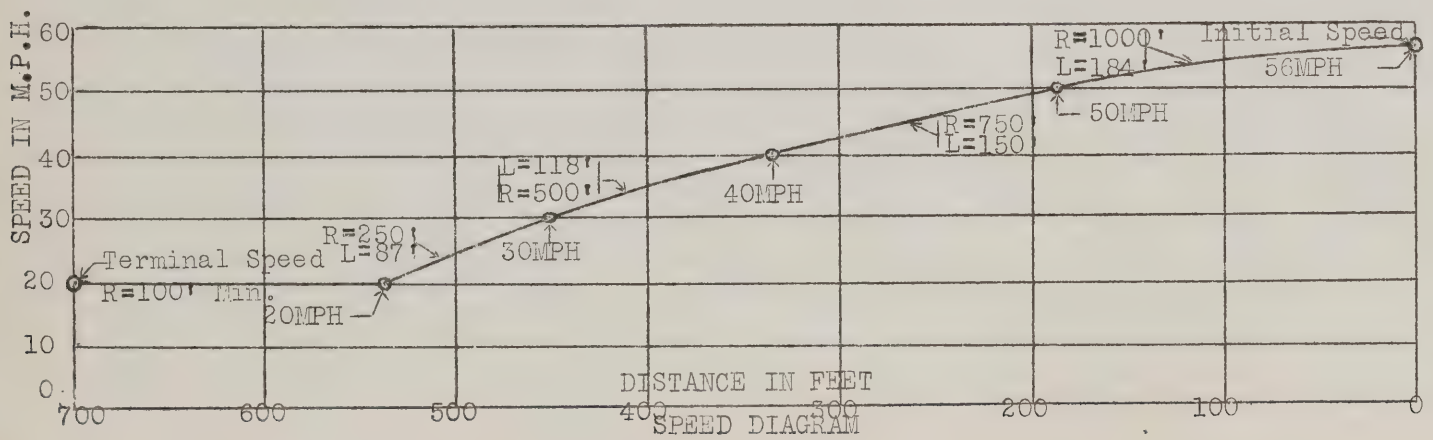


DIAGRAM OF FORCES ACTING ON CAR

LONGITUDINAL FRICTION COEFFICIENT F=0.20=6.44 FEET PER SECOND PER SECOND DECELERATION OF CAR



SPEED DIAGRAM

PENNSYLVANIA TURNPIKE COMMISSION
SPEED STUDY AT TURNOUTS
DESIRABLE LAYOUT FIG.19
April 20, 1939 Drawing No.M-3

understanding time 3 seconds, the letters of the sign must be of such a size that they can be easily read at a distance of 264 feet. In this case the sign may be said to be 3 seconds from the Motorist. The sign also should be set sufficiently in advance of a turnout for the motorist to make up his mind if he wishes to make the turn, for brake reaction time and for deceleration at a moderate rate to the speed required for the turn. If these functions require 5 seconds (440 ft. at 60 MPH) the turnout may be said to be 5 seconds from the sign. The fact that many direction signs have letters of improper design and size to be read and understood at as moderate a speed as 40 MPH, and that they are placed an improper distance in advance of turnouts is an indication that the factors are either unknown or that they have not been widely disseminated.

A very promising start has been made on an analysis of the effectiveness of highway signs by Dr. Forbes of the Bureau* and a test program to discover the fundamentals of letter legibility** was undertaken last Summer jointly by the Bureau and the Pennsylvania Turnpike Commission. These tests provided basic information on legibility not hitherto available and the results have been effectively utilized in the design of Turnpike Signs. It is hoped, however, that the program will be continued and enlarged until all the unknown factors relating to Signs have been determined so that this important phase of highway design may proceed with certainty.

PAVEMENT

The design speed will have an important bearing on the type of pavement selected, for experience during high speed runs on the salt flats of Utah showed that a motor vehicle "Takes to the air" when irregularities in surface are encountered. At the moment no one is thinking of highways speeds comparable with these test runs, but nevertheless surface smoothness should receive careful attention if the design speed is 70 MPH or upwards. Furthermore, it is quite as important

* Journal of Applied Psychology Vol XXlll, No. 6 p.669 (December 31, 1939)

** Highway Research Board, 1939 (Forbes & Holmes)

that the riding qualities should remain at a high level during the entire life of the pavement. It would be unthinkable to design a highway for high speed operation, and after spending large sums to accomplish that operation, select a type of pavement which would lose its riding qualities after a few years. Such a course would, in all probability, result in resurfacing long before the end of the otherwise useful life of the pavement.

In this connection, a test program to discover the maximum irregularities permissable in a pavement at various speeds would be most helpful and undoubtedly would result in large savings which might otherwise be wasted if improper pavement were used in express highways.

Because of the high traction requirements for speeds in excess of 60 MPH, the surface texture and skid proof properties of a pavement are important, and from the standpoint of safety it is fundamental that these properties remain uniform and consistent throughout every square yard of the surface.

GRADES

For the past 25 years there has been an almost ceaseless discussion of highway grades, but to date nothing concrete has been accomplished toward determining fundamental principles of motor transport on grades and relating economics thereto. As a matter of fact, there is an interesting disertation on the merits of low gradients in connection with horsedrawn vehicular traffic in the Department of Agriculture Report for the year 1866. In view of the voluminous discussion it is somewhat strange that more precise knowledge has not been accumulated which would permit a more accurate determination of the proper maximum rate of grade to use for any combination of traffic volume and character.

To be specific, no concrete engineering opinion has been formed on the proper operating cost for each class of vehicle on various rates of grade and the value of time which can justifiably be used in making an economic grade comparison.

It is true that manufacturers furnish certain parts of the operating data required but these data are based on new motors and loading in accordance with the rated capacity of the vehicle, while it is well known that practically all trucks overload beyond the rated capacity and a new truck motor ceases to be new after about one month's operation.

A promising analysis has been made of the general problem of highway economics by the Oregon Highway Department* and an excellently conducted series of tests**, (also a series by Public Roads Administration***) taken under actual commercial conditions on a normal public highway for the purpose of determining the effect of grades on fuel costs, have been made available to highway engineers. These data go a long way toward a solution of the grade problem but it would appear very desirable, for grade analysis purposes, to reduce operating costs to zero grades and delete from average operating figures the effects of stops, operation in congested traffic areas and poor alignment. It is also believed that the rate of grade will have an effect on the cost of oil, tires and maintenance.

The value of time for both passenger cars and trucks will vary in different parts of the country and between strictly rural areas and metropolitan areas with a heavy industrial development. In addition, truck speed and operating costs on grades, in all probability, vary depending on the gear ratios used in any particular territory.

One of the most controversal aspects of the situation, from the standpoint of the public, is the extent to which money should be expended to reduce grades for the benefit of trucks and the determination of an economic balance of expenditures based on the relative requirements of and benefits derived by trucks and passenger cars. The attitude in the past in many quarters has been that trucks were in business for private profit and that it was not in the public interest to expend governmental funds for the benefit of private interests. Consequently, highway grades have been determined largely by the operating requirements of passenger cars. Inasmuch as the general public eventually pays the full cost of transportation when purchasing goods, this policy has not resulted in an economic grade design.

*Bulletin #7, Oregon Highway Department (1937 - revised 1938)
**Bulletin Oregon Highway Department (1937)
***Studies of driver behavior and Vehicle Performance (1939)

The railways*, on the other hand know precisely the speed-load factor for every type of motive power available and in fact can calculate the sums of money that can profitably be expended for grade reduction for a given volume (tonnage) of traffic. They prepare speed charts for each division and know the length of travel time required for each motive power unit hauling an economical load. The reverse is true on the highways.

For the first time in the history of motor transpot it will be possible, when the Turnpike is opened for operation, to determine the extent that the public and truck operators will be willing to pay in cash directly for superior highway facilities, and when this is demonstrated, the principle of designing for over-all benefit to all classes of traffic should receive universal acceptance and an unexampled opportunity will be presented by the Turnpike to accumulate authoritative data on operating characteristics and costs of trucking units. In addition, the character and volume of traffic willing to pay for deluxe highway service can be determined by the actual tolls which are paid for the privilege of using the Turnpike and it may be stated with certainty that the amount of tolls paid by truckers in comparison with the tolls paid by passenger cars will represent the minimum economic benefits to such truckers. It will be possible with the toll collection system adopted, to obtain the precise figures paid in by trucks for each weight classification. It is interesting to note that the Traffic Engineers' report on the volume and nature of traffic predicted to use the Turnpike and the revenue therefrom, shows that 77% of the volume will be passenger cars and 23% trucks but that 54% of the revenue will be derived from trucks, while only 46% will be derived from passenger cars.

A few illustrative examples of the grade problem are given in order to present the situation with respect to grades more clearly. The values assigned to the various factors were taken from various sources but as previously noted very few of the factors have been authoritatively established on a sound basis. The examples are crude and should

*Economic Theory of Railway Location by Arthur M. Wellington.

not be used as a guide for analyzing an actual problem since no attempt has been made to break down operating costs of trucks into gross weight groups or to evaluate properlythe influence of descending grades, but it is hoped that they will provide an insight into the problem of grade economics as well as indicate the factors of operating cost and time which are not precisely known.

Example No. 1.

If it is assumed that it would be possible to project a straight ("air-line") route (Route B) with no curves from the starting point of the Turnpike at Middlesex to the terminus at Irwin, a distance of 135 miles will result in comparison with the actual length of 160 miles (Route A). If it is further assumed that in order to achieve a straight route it would be necessary to use 20 miles of 5% adverse grades (adverse to both directions) in comparison with the 3% grades actually adopted and that in the remaining distance the grades would conform to Turnpike grades; and that the average speed of an average weight truck over the Turnpike (Route A) would be 35 MPH while over the alternate Route (Route B) the average speed would be 28 MPH. It is assumed that the average speed of passenger cars over both routes would be 60 MPH. Under these assumptions the total time of travel is as follows:

Comparative Time of Travel

Line A (160 miles)		Line B (135 miles)	
Type of Vehicle	Time in Hours	Type of Vehicle	Time in Hours
Passenger Car	2.67	Passenger Car	2.25
Truck	4.57	Truck	4.82

The remaining assumptions are as follows:

Cost of average weight truck operation on grades up to and including 3%	10¢ per mile
Cost of average weight truck operation on 5% grades	13¢+per mile
Cost of passenger car operation	2¢ per mile
Value of passenger car time	30¢ per hour
Value of average weight truck time	$3.00 per hour

The assumed cost of truck and passenger car operation includes only the items of gasoline, oil, tires and maintenance. A calculation reveals the following:

Economic Cost Comparison

Route A (160 miles)			Route B (135 miles)		
Passenger Cars			Passenger Cars		
Time	(2.67 x .30)	$ 0.80	Time	(2.25 x .30)	$0.67½
Operating Cost	(160 x .02)	$ 3.20	Operating Cost	(135 x .02)	2.70
Sub Total		$4.00	Sub Total		$3.37½
Trucks			Trucks		
Time	(4.57 x 3.00)	13.71	Time	(4.82 x 3.00)	14.46
Operating Cost	(160 x .10)	16.00	Operating Cost	(115 x .10)	11.50
			Operating Cost 5% grade	(20 x .13)	2.60
Sub Total		29.71	Sub Total		28.56
TOTALS		$33.71			$31.93½

The traffic engineers predict that 1,000,000 passenger cars and 300,000 trucks will use the Turnpike during the first year's operation, which with the above assumptions, indicates the annual operating savings for Route B (which utilizes 20 miles of 5% grades) over Route A would be as follows:

Annual Savings to vehicles on Route B

Passenger Cars	(1,000,000 x $0.625)	$625,000.00
Trucks	(3,000,000 x 1.15)	345,000.00
		$970,000.00

The saving of $970,000.00 annually would theoretically justify an expenditure of $17,000,000.00 more for Route B than for Route A, assuming full interest at 3-3/4% and amortization of principal within 30 years. But as a matter of fact, if the topography of Route B were similar to Route A, the former could be constructed, due to being 25 miles shorter, for (345,000 x 25) $8,625,000.00 less than the the latter; and in addition, a yearly saving in maintenance, policing and snow removal costs would result, which is estimated to amount to $90,000.00 (25 x $3600).

Example 2

If a problem is encountered where it is necessary to climb up to a plateau and it is possible to choose between a route (Route D) 10 miles long on a 5% grade and a route (Route C) 16.67 miles long on a 3% grade (Both routes reach the same elevation) the following crude analysis indicates the comparative economic operating costs, utilizing the same assumptions noted above, except that truck operation on composite 5% grades will cost 12 cents per mile.

Comparative Time of Travel

Route C (16.67 miles)		Route D (10 miles)	
Type of Vehicle	Hours	Type of Vehicle	Hours
Passenger Car	0.2778	Passenger Car	0.1667
Truck	0.605	Truck	0.525

Economic Cost Comparison

Route C (16.67 miles)			Route D (10 miles)		
Passenger Cars			Passenger Cars		
Time	(0.2778 hrs. x $.30)	$0.083	Time	(0.1667 hrs. x.30)	.05
Operating Cost	(16.67 miles x $.02)	0.333	Operating Costs	(10 miles x.02)	.20
Sub Total		$0.416	Sub Total		$0.25
Trucks			Trucks		
Time	(.605 hrs. x $3.00)	1.81	Time	(.525 x 3.00)	1.57
Operating Cost	(16.67 miles x .10)	1.67	Operating Cost	(10 miles x .12)	1.20
Sub Total		3.48	Sub Total		2.77
TOTALS		$3.896	TOTALS		$3.02

Annual Savings to Vehicles on Route D

Passenger Cars	(1,000,000 x $0.166)	$166,000
Trucks	(300,000 x $0.71)	$213,000
	Total	$379,000

The saving in operating costs of $479,000, annually would theoretically justify an expenditure of $6,750,000, more for Route D than for Route C, but as a matter of fact, if the topography of the two routes were similar, the former could be constructed for (345,000.00 x 6.67) $2,300,000.00 less than the latter; and in addition there would be an annual saving in maintenance, policing and snow removal amounting to (6.67 x $3600) $24,000.

Example 3

This example is presented to illustrate the economic value of constructing tunnels through mountains of the ridge type where there is a sufficient volume of traffic. The average speed for vehicles climbing the mountain were clocked during a state traffic survey. The equivalent values for truck weight and distance were applied to a nearby section of the Turnpike where it will pass under the same mountain in a tunnel. The following is an economic comparison of a tunnel route (Route F) versus an over-mountain route (Route E). The horizontal distance for each route is the same, but Route E required traffic to climb to an elevation of 1420 feet above the starting point and then descend 1200 feet, whereas the rise and fall on the tunnel route (Route F) is negligible.

Comparative Time of Travel

All times given are based on Composite Speed

Route E (7.9 miles)				Route F (7.9 miles)	
Easterly Portion		Westerly Portion		Complete Section	
Type of Vehicle	Hours	Type of Vehicle	Hours	Type of Vehicle	Hours
Passenger Car	0.067	Passenger Car	0.076	Passenger Car	0.143
Truck	0.284	Truck	0.187	Truck	0.204

It is assumed that passenger cars will travel both routes at the same average speed since a comparison of the slow down effect due to alignment is not under consideration.

Economic Comparison of Tunnel Route and Over Mountain Route

Route E (7.9 miles) (Over-mountain route)		Route F (7.9miles) (Tunnel route)
Easterly Portion (3.7 miles)	Westerly Portion (4.2 miles)	Complete Section (7.9 mi)
Passenger Cars	Passenger Cars	Passenger Cars
Time (0.067x.30) $0.020	Time (0.076x.30 $0.023	Time (0.143x.30 $0.043
Operating (3.7x0.0224)0.083	Operating (4.2x0.0216)0.091	Operating (7.9x.02)0.158
Sub Total $ 0.103	Sub Total $0.114	Sub Total $0.201
Sub Total both sections $0.217		Sub Total $0.201
Truck	Trucks	Trucks
Time (0.284x3.00) $0.850	Time (0.187x3.00) $0.561	Time (0.204x3.00) $0.612
Oper. (3.7x0.193 0.715	Oper. (4.2x0.171) 0.718	Oper. (7.9x.10) 0.790
Sub Total $1.565	Subt Total $1.279	Sub Total $1.402
Sub Total both sections $2.844		Sub Total $1.402
Totals	$3.061	$1.603

Annual Savings to Vehicles on Route F

Passenger Cars	(1,000,000 x $0.016)	$ 16,000.00
Trucks	(300,000 x $1.442)	432,600.00
	Total	$ 448,600.00

From the vehicle operating savings must be deducted the cost of tunnel maintenance, policing, attendance, ventilation and lighting. This is estimated to cost $20,000.00 per year, so that the net savings realized from vehicle operation amounts to $428,600.00.

The net saving in annual operating costs justifies an expenditure (provided the assumptions are correct) of $7,650,000.00 for constructing a tunnel and this may be compared with a comparative actual cost of a Turnpike Tunnel of $1,750,000.00.

The above figures relating to over-mountain operation are undoubtedly lower than actual costs because the Oregon data indicates higher truck speeds in descending 5½% and 7.3% grades than actual clocking on Tuscarora. It is likely that this discrepancy is caused by the fact that the grades used in the example were average grades and that the maximum grades for several miles were in the neighborhood of 9%. For reasons of safety, truckers descend 9% grades in low gear and often travel slower when descending than when ascending. Therefore, it is quite possible, if a really accurate economic analysis were made, that actual operation savings would justify a capital investment for a tunnel greater than $7,650,000.00.

In examples 1 and 2 the point will be noted that it is more economic to utilize 5% grades instead of 3% grades when rise and fall or distance is not reduced on the 3% route. This immediately raises the question as to what maximum grade is the most economic for given topography and volume and character of traffic. Although there is no conclusive factual evidence to prove the opinion, it is the writer's belief that any increase in grade above 5% introduces a sufficient element of hazard to traffic when descending such grades during icy weather to add an appreciable factor in the form of accident losses to the economic cost. Whether the economic losses due to accidents are of sufficient magnitude to warrant limiting maximum grades to 5% is of course unknown at the present time.

Personal opinion, however, does not bear sufficient weight to justify the expenditure of vast sums of money to construct highways on low grades* and it is therefore imperative that authoritative factual data be accumulated before any large program of express highways is instituted.

*So far as the Turnpike situation is concerned, the general route of the Old South Penn Railway was located so as to pass around the ends of mountain ranges with as few tunnels as possible. Any other more direct route would have encountered more ranges and consequently necessitated more tunnels. Since the South Penn was located on 1% grades with a maximum of 1.9%, it was nearly as economical to design the Turnpike on 3% grades as on 5% and the topography is such that very little shortening of distance could have been accomplished by the use of steeper grades.

UNIFORM OPERATING CONDITIONS

Many Highway accidents are traceable directly to non-uniform inconsistent operating conditions and such non-uniformity also has an influence on the average safe speed that can be maintained; which in turn has an effect on the economic effectiveness of the project. The motorist does not deliberately set out to kill himself or to destroy a valuable piece of property, his car, but oftimes, he is suddenly confronted with the unexpected in road conditions and an accident results. This is particularly true where high speed operation is involved, inasmuch as human reaction time is relatively constant whereas as the speed is advanced less time is available to copy with operating hazards, and it is therefore essential for the engineer to provide the motorist with operating conditions as consistent and uniform as possible at every point on the highway.

If a design speed is adopted, which is high enough to accommodate the reasonable speed potentials of the modern motor car, and curvature as well as daytime and night time sight distance is designed consistently for this speed, utilizing conservative factors, a considerable step will have been made in providing uniform operating conditions. In the event that it becomes necessary to change the design speed because of physical or economic conditions such a change should be made gradually over a distance of several miles, and in addition, the motorist must be warned with adequate signs that such a change is being made. In designing such a transitional section it must be remembered that a car traveling at a rate of 60 MPH will cover 5 miles in 5 minutes. Because of the relatively short time in which a modern motor car can cover distance, it would appear reasonable to limit changes in design speed to sections not less than 50 miles in length.

In addition to the provision of a uniform design speed and attendant uniformity in sight distance, it is of great assistance to the motorist if uniformity in grade conditions are provided. It is very disconcerting and often hazardous when operating at night or during periods of poor visibility to run into a section with short choppy grades after having operated over long smooth grades. This is particularly true if the choppy grades are combined with horizontal curvature. If it is necessary to introduce rolling grades because of economical considerations, a transition section should be utilised between the smooth grade section and the choppy grade section, with the profile tangents shortening successively until reduced to the desired length. When designing profiles it is well to bear in mind that, from the standpoint of the motoring public, the projects with profile tangents a mile or more in length apparently receive the greatest approbation.

There has been considerable misunderstanding respecting the relative economy of rolling grades and long smooth grades, it being nearly the universal opinion that the former are always more economical than the latter. It is probable that this opinion had its origin in the days when earthwork was balanced out within 500 or 1000 foot sections and haul beyond the limits of a grading section was paid for at a bid price per station yard. The shortness of the grading sections was necessitated by the grading equipment then in use, which was slow and limited in capacity. Today, with modern grading equipment, the economical haul limit has increased so that grading sections a mile or more in length are utilized and this necessitates a revision in ideas respecting grade economy.

The first grading contract let on the Turnpike due to the extreme necessity for haste,was designed with a considerable number of rolling grades. As the design progressed from one contract to another this situation gradually improved and in a number of instances where time was available for comparative estimates it was discovered that long smooth grades were actually more economical than rolling grades. The earthwork on the Turnpike was balanced within grading sections approximately one mile long and the very low average bid price ($0.43 per cubic yard) for unclassified excavation indicates that one mile grading sections is quite economical when there is sufficient volume of material to justify utilizing modern heavy duty excavating equipment.

Other features which have an important bearing on uniform and consistent operating conditions include uniformity in width of the graded roadway, width of bridges, treatment of bridge parapets, position and design of guard rail,stabilization of shoulders, the size, design, location of signs and consistency in sign message. Pavement markings should also be consistent, following a definite and standard pattern so that the starting point and position will in all cases bear the same relation to other design elements.

ACCESS FACILITIES

As previously stated, access and exit facilities should be spaced reasonably far apart on high speed routes so as to minimize the frequency of the hazardous impact effect of exiting or entering traffic. Traffic analysis to determine peak hours flows at each point of interchange are essential in order that the poi may not be spaced so far apart as to concentrate a heavy flow onto or off of the facility at one point. Such concentration will defeat the objective of minimum turbulence for it will be preferable to have the interchange points closer together than to concentrate a heavy flow, with resultant congestion and hazard, at one point. Convenience to the public and consideration of economic traffic distribution throughout the territory served should receive careful consideration and analysis.

Modern conception of traffic requirements demands that there should be no

left turns or cross traffic at interchange sites (or elsewhere) and that the design be premised on allowing sufficient time and space to accommodate the potentials of the motor car and the vagaries of the operator. Insofar as possible the layout should be so arranged that the driver will not have more than one decision to make at a time and simplicity of design should be the keynote. Design should be such that if possible it will be physically impossible for a driver to perform a wrong movement and conditions should be so arranged that right movements will be encouraged by making them obvious, natural, simple, pleasant of performance and comfortable, while the reverse should apply to wrong movements. The physical requirements of the vehicle, the inclinations, preferences, weaknesses and physchology of the driver are positive design elements and constitute the "materials" of construction.

The provision of deceleration and acceleration lanes is accepted as standard practice today but the length and design of such lanes is quite unsettled. Standard treatment is not possible or desirable since the design should vary depending on the design speed of the highway, the volume and character of traffic and the grade conditions. It should be evident that where mixed traffic is to be accommodated and an acceleration lane enters the main highway where there is a heavy adverse grade, the acceleration lane should be lengthened sufficiently to minimize the retarding effect of the grade. Correspondingly where traffic forecasts indicate a heavy concentration of entering traffic the acceleration lanes should be lengthened to permit increased opportunity for this flow to gradually work into the main traffic stream. Unless additional lanes are added to the main highway, not more than a single lane of inflowing traffic should be permitted to impinge on the main roadway. Many designs are faulty in this respect since frequently a two lane ramp is brought into two one directional lanes of main traffic. In such a case the entire main roadway (one direction) is completely blocked each time two vehicles enter abreast from the ramp, and since the speed of the entering vehicles is usally less than on the main highway, a serious accident hazard is produced in addition to the adverse effect it has on traffic movement.

Figure 20 shows a typical traffic Interchange on the Pennsylvania Turnpike.

SYLVANIA TURNPIKE

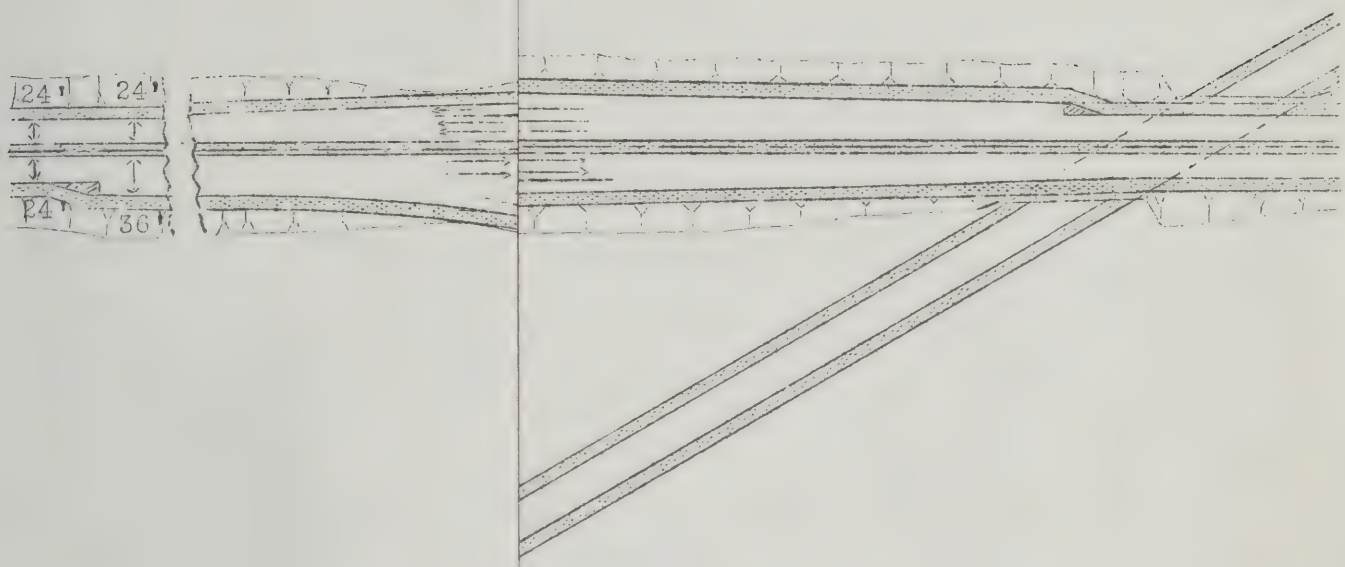


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FIG. 20

Pennsylvania Turnpike Commission
PENNSYLVANIA TURNPIKE

TYPICAL INTERCHANGE

November 7, 1939

M-34

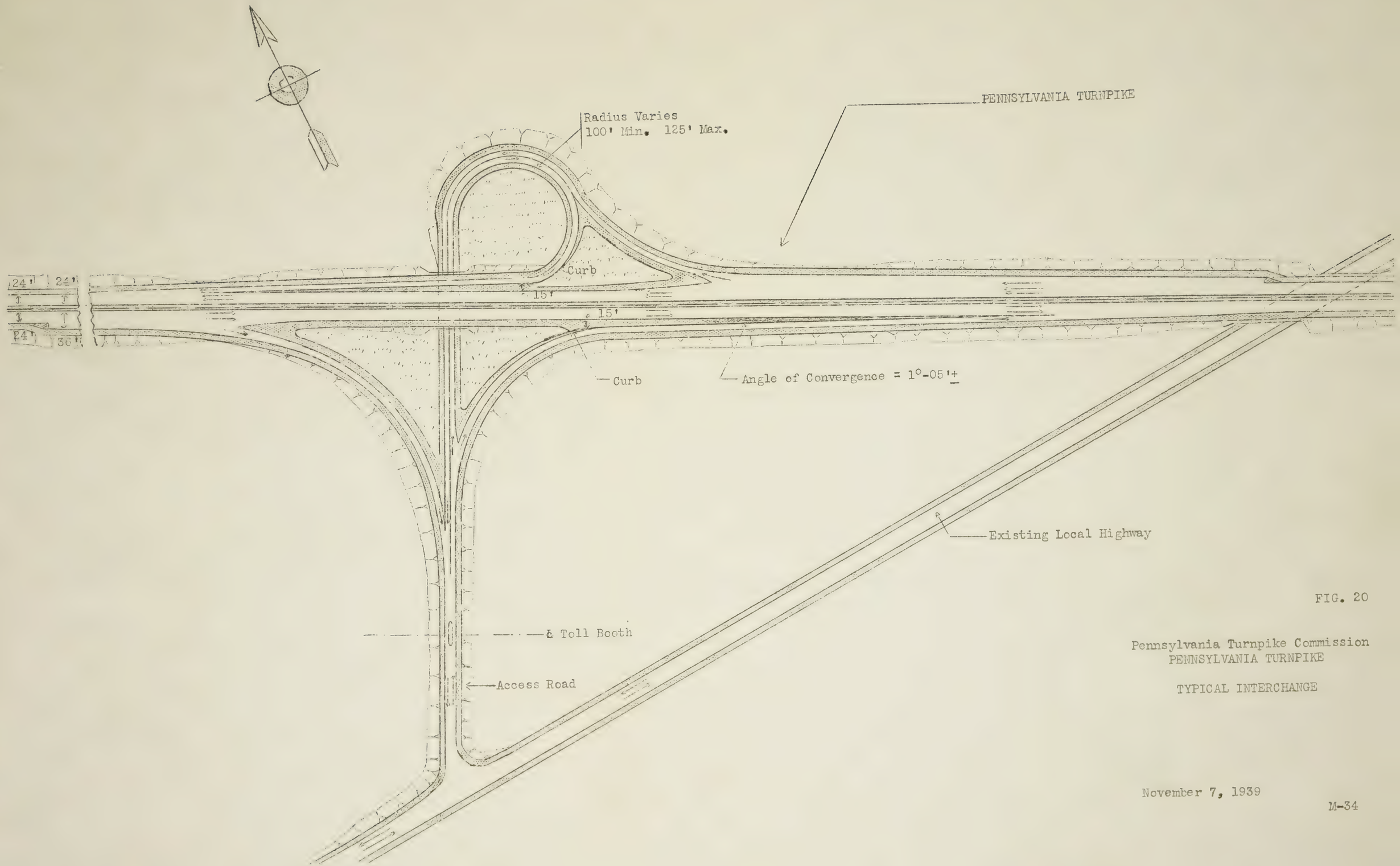

FIG. 20

Pennsylvania Turnpike Commission
PENNSYLVANIA TURNPIKE

TYPICAL INTERCHANGE

November 7, 1939

M-34

The acceleration and deceleration lanes are 1200 feet in length and the turnouts are designed for an initial speed of 56 MPH as previously described and shown in Figure 19. The motorist has only one choice and decision to make at a time. The presence of the Interchanges will be made known to the motorist in advance at points two miles, one mile, one half mile, on reaching the deceleration lane with a "Keep Right for Exit" sign located 900 feet in advance of the turnout, a routemarker with direction arrow 400 feet in advance of the turnout and a confirming marker a short distance down the ramp. The main line traffic is given a confirming marker just beyond the turnout. "No Left Turn" signs are placed at places where motorists may be tempted to make left turns, but if such signs are not effective, curbs will have to be installed to form a positive barrier.

It is interesting to note that the average cost of an Interchange on the Pennsylvania Turnpike complete without ticket booths and equipment amounts to $143,000.00; and that the estimated cost of operation with ticket attendants and maintenance, exclusive of lighting, will be $8,000.00 per annum each.

FINANCIAL COMPARISONS

In many quarters there appears to be a confusion and misconception regarding the relative cost of the various elements which go to make up a completed highway project. Protests are frequently made that this feature or that feature, while very desirable, cost too much money and cannot be afforded. In the words of a well known politician, "Lets look at the record".

An analysis of cost of the various construction features on the Pennsylvania Turnpike may be of interest and Table II has been prepared on the basis of the figures that are available to date. Because the Turnpike is not completed and final payments not made, the figures must be considered as approximate, although the error in percentage in relation to the whole cost will be very small. The table was prepared on the basis of the net highway length and costs without the tunnels because it was felt that a better picture of express highway costs would be obtained if the tunnels were left out. Certain costs, whether tunnels were present or not, were included

TABLE II

Approximate Turnpike Costs Without Tunnels.

Item	Cost Per Lin. Foot of Turnpike	Percentage of Total
(1) Clearing (1)	$0.452	0.69
(2) Right of Way**	4.154	6.35
(3) Relocation of Utilities	0.403	0.62
(4) Restoration of Highways	0.434	0.66
(5) Bridges	8.453	13.02
(6) Drainage	3.842	5.88
(7) Excavation	12.607	19.40
(8) Pavement	14.452	22.20
(9) Roadway Shaping	0.093	0.01
(10) Subgrade	0.510	0.79
(11) Shoulders	0.314	0.48
(12) Interchanges	1.874	2.87
(13) Ticket Offices	0.212	0.32
(14) Fare Equipment	0.045	0.01
(15) Maintenance Buildings	*0.238	0.36
(16) Maintenance Equipment***	*0.475	0.72
(17) Administration Building***	*0.119	0.02
(18) Erosion Control***	*$0.929	1.42
(19) Fencing	* 0.929	1.42
(20) Guard Rail	0.728	1.12
(21) Signs	0.036	0.01
(22) Pavement Markings	* 0.025	0.01
(23) Communications***	* 0.356	0.54
(24) Testing	0.119	0.02
(25) Engineering	5.240	8.10
(26) Administrative & Legal	1.910	2.92
(27) Interest During Constr.(z)	1.078	1.66
(28) Interest During First Year's Operation (z)	1.486	2.28
(29) Bond Discount (z)	3.360	5.15
(30) Contingency Surplus	0.620	0.95
Total Cost Per Linear Foot of Turnpike	$65.493	100.00

(1) Includes cleared areas where no clearing was required.

* Estimates only. ***Not approved for construction to date

**Estimates only as very little acquisition consummated to date. Includes $2,000,000 paid for South Penn R.O.W. Interes during first year's operation required by Trust Indenture

(z) Based on only 55% of total cost, on account of 45% P.W.A. grant.

on the basis of the entire length of project, since their frequency is dependent on total length regardless of whether a portion of the length was composed of tunnel.

The cost items of a completed highway project naturally group themselves into certain broad divisions of a functional nature. For purposes of discussion these are listed with the percentage comparisons based on estimated Turnpike costs (without tunnels), in Table lll.

Items 8 to 11 inclusive, bearing a relation of 28.8% to the whole cost, may be considered broadly as fixed charges regardless of width, alignment and grades.

Items 5 to 11, bearing a relation of 72.3% to the whole cost, may be considered broadly as fixed charges regardless of alignment and grades.

Items 1, 2, 3 and 5 to 11 inclusive, bearing a relation of 80.6% to the whole cost, may be considered broadly as fixed charges regardless of grades.

There has been from time to time in the technical press an outcry respecting the excessive cost of wide rights of way and in many built-up areas right of way costs are sufficiently large to be significant. But for rural areas and where the location generally traverses the outskirts of populated areas such as is the case on the Turnpike, right of way costs bear out a rather modest relation to the whole cost of the project. In the case of the Turnpike, if the South Penn Railway payment is deducted, it amounts to 2.7%.

Yet the right of way is the only item with a life which can be infinity. It therefore appears, simply in the interest of sound investment, for the alignment, grades and width of the improvement to be such that that the life of the right of way will be infinity. If either of these three elements (alignment, grades, width) are such that a relocation is required in the future the entire capital investment is jeopardized, except for the amounts depreciated from the capital account. If the right of way is of adequate width and on proper alignment and grades, the investment in all other items is conserved and it only becomes necessary to replace such portions as and when their useful life ceases, due to age and wear rather than from obsolescence.

TABLE III

BROAD DIVISIONS OF HIGHWAY COSTS

Item #	Classification	% of total
(1)	Clearing and Grubbing	0.69
(2)	Right of Way* (Minimum R.O.W. 200' wide)	6.35
(3)	Relocation of Utilities and Highways	1.28
(4)	Excavation	19.40
(5)	Drainage	5.88
(6)	Bridges	13.02
(7)	Surfacing (Includes Roadway Shaping, Subgrade, Shoulders Guard Rail)	24.60
(8)	Traffic Interchanges (Includes Bridges, Grainage, pavement, etc.)	2.87
(9)	Operation Facilities (Includes Adm. Bldg., Maintenance Bldgs. Ticket Offices, Fare Collection Equip., Maintenance and Snow Removal Equip., Police cars, Communication System, Signs, Fencing, Erosion Control and Traffic markings)	4.83
(10)	Engineering, Administration and Financing (Includes Engineering, Equipment, Transportation, Consulting Fees, Testing, Inspection, Interest During Construction**, Bond Discount**, First Year's Interest***.)	20.13
(11)	Contingency Surplus	0.95
	Total	100.00

* Including $2,000,000.00 paid for South Penn Railway right-of-way.

** Based on only 55% of total cost, on account of 45% P.W.A. Grant.

*** Required by Trust Indenture.

As a general rule, new right of way cannot be obtained for nothing, so that the difference in cost between the absolute minimum in width and that which is desirable not only for present needs but also for future possibilities, is not as great as many believe. For a grade separated highway such as the Turnpike, access cannot be had to property from one side of the highway to the other. Often it is cheaper to buy the portion of a property cut off from the rest rather than build an access structure; and the costs of this type of acquisition bear no relation whatever to the width of the right of way; the same conditions in more or less degree apply to consequential damages and to the cost of acquisition. In general, therefore, it may be said that in rural areas restriction of access and consequential damages will not be influenced by the width of the right of way, and it may be concluded that the actual cost of extra width right of way in relation to the total project is very small indeed.

Because highway engineers are facing today the unfortunate consequence resulting from poor alignment and narrow rights of way, it is sincerely hoped that the engineer will profit by that lesson and promote the idea of purchasing ample rights of way for express routes.

On a project such as the Pennsylvania Turnpike, traversing as it does a mountainous territory, the quantity (26,000,000 cubic yards) and cost of excavation is a considerably more important item than on routes located in more favorable topography; and yet the item of excavation amounts to only 19.4% of the total cost of the project. Next to the right of way, excavation is the most permanent item in highway construction. Properly maintained it will last for centuries provided it does not have to be abandoned because of obsolescence in alignment and grades. It will be noted that the proportionate cost of pavement, which has a relatively short life of 20 years, is more than the cost of excavation, but unfortunately expensive pavement is often laid on poor alignment and grades in order to save ...avation and later has to be abandoned before it has served its useful life because ...ignment or grades have become obsolete.

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The difference in cost between poor and good alignment and grades will seldom if ever amount to more than two or three percent of the cost of express highway projects, and yet the entire capital investment is often rendered ineffective because of a desire to cheapen excavation costs. This policy does not appear to be in the interest of sound, long-range economics.

It is believed that a study of Tables II and III will indicate that mere cheapness in first cost is not always the soundest course in the long run, particularly in those items where a moderate increase on the percentage of cost, as compared with the whole, will assure the entire remaining investment against early obsolescense. If the project must be cheapened a fertile field for constructive work is available in developing ways and means to decrease the costs of financing and interest burden and this latter item may be expected to be larger on the average project than on the Turnpike because it is doubtful if many undertakings will be carried through with a 45% P.W.A. grant and with the extreme speed that has characterized the progress of the Turnpike.

CONCLUSION

The present problems and unknowns facing the highway engineer and economist are solvable and will be solved when their existence and importance is recognized and when men or organizations can be found with the initiative and energy to undertake the labor and expense which must be expended in their solution. In conclusion the hope is expressed that this rough outline of some of these problems will, in some measure, stimulate such research and investigation so that express highway design may proceed with certainty and so that it will be based on fundamental safety and broad economics.